

# JOURNAL OF THE AMERICAN WATER WORKS ASSOCIATION



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*by the*

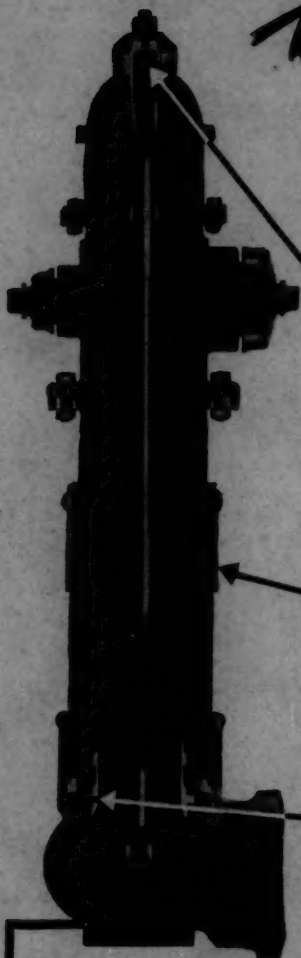
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# Frost Strikes

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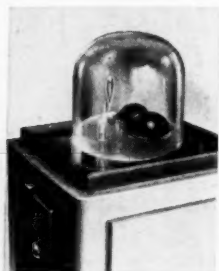
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Vol. 31

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## Service Lives of Water Mains

As Indicated by a Seventy-Five Year Retirement Record

*By Reeves Newsom and E. H. Aldrich*

THE lack of adequate knowledge of the service lives of main pipe has profoundly affected the water supply industry. The proper length of life for bonds issued by municipalities has been a guess because of it. The accumulating of a reserve for the replacement of the pipes at the end of their usefulness and the setting of rates for water service have, to a large extent, been dependent upon the arbitrary selection of service life figures for pipe with results generally unsatisfactory to all parties. The designs of distribution systems and transmission mains have often been economically unsound because facts were not available as a basis for calculations.

Some of the various items of property making up a water works system are relatively short-lived and their average duration of time in service can be and has been determined within reasonable limits. However, in the case of the largest elements of property, the transmission and distribution mains, such is not the case. These elements rarely amount to less than 50 per cent of the total value of the complete system and in many systems approach 80 per cent of this total. It is evident that the correct estimation of the consumption of value of these elements, in view of the necessity of replacing such value

A paper presented on June 14, 1939 at the Atlantic City Convention by Reeves Newsom and E. H. Aldrich, Engineer-Consultants, New York, N. Y.

through the rates to be charged, is in many cases a paramount factor in the cost of furnishing water service.

In no other utility business is one element of a plant of such importance. Whether a plant be publicly or privately owned and operated does not influence its importance for in either case the results in the final analysis are the same.

In the past, so far as is known to the authors, there has not been a factual study of importance, which has determined from actual long time records the service lives of mains in water service. The question naturally arises as to why this should be so. The answer is that very rarely does a water utility have actual records of mains installed, mains retired and the dates at which these changes took place, over a sufficient period of time to approach an adequate basis for a serious study seeking a determination of the service lives of mains. Even when, as in the case herein reported, there are substantially complete records covering a period of seventy five years, the duration of time is too short for a completed study. Nevertheless the indications shown by such a study are of sufficient importance to justify making it.

### Conditions in System Studied

An investigation such as this is pertinent and of value only if the basic accuracy of the data and the extent and accuracy of the study can be shown to justify the application of its results.

It is particularly fortunate that a system of adequate records covering seventy five years is available in a large and important water works system of pre-eminent standing in the United States. Such a system is the municipal water works of the City of Springfield, Massachusetts.

These works now serve the City of Springfield and communities of West Wilbraham, Agawam, East Longmeadow, Longmeadow, Southwick, Provin Mountain, Mundalo, Ludlow, Wilbraham and, in emergency, West Springfield, with a combined population of over 150,000. The plant has operated as a municipal system since 1872. It has had excellent management and engineering throughout this period. It can be said to have survived all the elements of risk, good and bad fortune and more than ordinary changes and has emerged as one of the best of the water systems of this country.

The adequacy and condition of the various elements of its property are of such high quality that no recommendation for improvement

was advanced by the National Board of Fire Underwriters in the latest report on this system dated February, 1938. In the summary the report reads:

*Water Supply*—Municipal works; management efficient. Records complete. Filtered gravity supply from distant source, adequate and dependable. Single supply line in part, mainly offset by good service record, considerable distributing storage and a supply available at lower pressure from a former source. Consumption moderate. Services metered. Pressures high and well maintained under heavy draft. Distribution in one service. System strong in arteries, secondary feeders and minor distributors. Mains in good condition. Gate valves well spaced and maintained. Hydrants well distributed and in excellent condition."

The National Board of Fire Underwriters in their conclusion say:

*"Distribution System*—In conjunction with two large supply mains from the distant distributing storage, arteries are sufficient in size and number and secondary feeders are generally well arranged for most of the system. Minor distributors are mainly 8-inch and the considerable amount of 6-inch is mainly located where pressures are high. Dead ends are avoided where practicable. Interior condition of mains is good. The system has large distributing strength and provides satisfactory quantities for protection."

To have achieved such a status, replacements of pipe having even a minor degree of inadequacy have taken place in an amount far beyond the experience of most water systems because it is recognized that in many respects the requirements set up by the National Board of Fire Underwriters are ideals toward which management of water systems work. To have satisfied all such requirements in a single system is, so far as is known to the authors, almost without precedent.

The population of Springfield according to the United States Census is as shown in table 1 and also on fig. 1. Prior to 1843, when the population was about 11,000 the people of Springfield depended upon individual wells and springs for their water requirements. In June, 1843, Charles Stearn built a reservoir at the site of the present Lombard Reservoir and laid about 3 miles of log pipe lines from 4 to 7 inches in diameter to serve a portion of the built-up section.

On May 10, 1848 a charter was granted, and in June of that year the Springfield Aqueduct Company was organized. The system started by Charles Stearn was purchased. The supply system was augmented by the construction of the Heywood, Stebbins, Chapin

and Worthington Reservoirs and the distribution system was increased to approximately 11 miles of mains, still using wood for pipe material. Approximately 700 families were served with water at this time.

In February, 1860 action was started toward the development of a municipal system. The City Aqueduct Company in a contract with the City dated September 10, 1860 commenced operation with a small spring collection system. In the following year, however, its rights were bought by the City as a result of objections from citizens relative to damage to individual well systems.

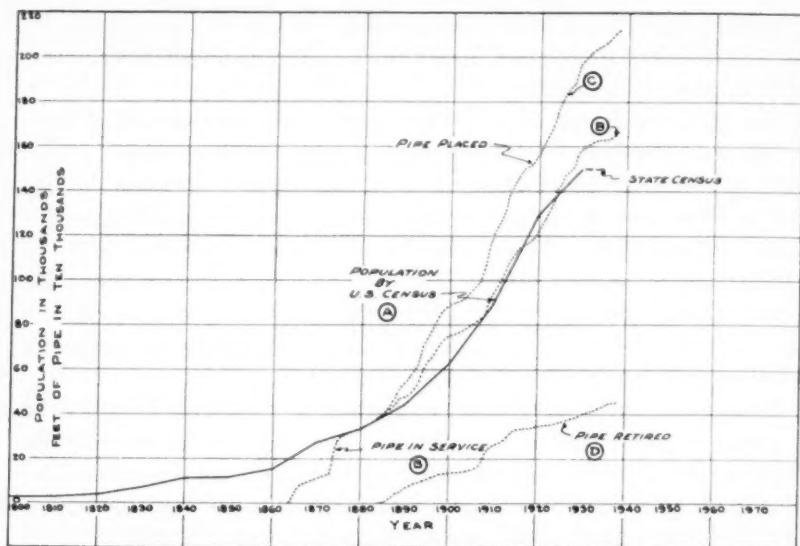


FIG. 1. Population and Pipe Growth, Springfield, Mass.

In 1864 the Springfield Aqueduct Company began replacing the original wood pipe with the Goodhue-Birnie wrought-iron cement-lined and coated pipe invented by Jonathan Ball and common to this locality during this period. The records of pipe installations and retirements started at this time.

From 1864 to 1870 the wood pipe was all replaced by cement pipe; Lombard Reservoir was reconstructed and enlarged; and the East and West Van Horn Reservoirs constructed. Minor sources and the Connecticut River had to be used to augment the supply for several periods in the early 1870's but these were temporary expediences.

The Springfield Aqueduct Company was purchased by the City on

July 1, 1872, but the system was not taken over until June, 1873. Immediately steps were taken to develop a new and larger supply, and during 1874 and 1875 the Ludlow Reservoir east of the City was constructed, across Broad and Axe Factory Brooks. About this time the city discontinued the installation of the cement-wrought-iron pipes and turned to cast iron as the sole pipe material for all except temporary domestic service pipe of small size wrought-iron pipe. Some large steel mains were later used in the supply and transmission elements of the property but in general cast-iron pipe was used from 1875 on.

TABLE 1  
*Springfield, Mass. Population Growth by U. S. Census*

DATE	POPULATION	INCREASE	% INCREASE
1790	1,574		
1800	2,312	738	46.9
1810	2,767	455	19.7
1820	3,914	1,147	41.5
1830	6,784	2,870	73.3
1840	10,985	4,201	61.9
1850	11,766	781	7.1
1860	15,199	3,433	29.2
1870	26,703	11,504	75.7
1880	33,340	6,637	24.9
1890	44,179	10,839	32.5
1900	62,059	17,880	40.5
1910	88,926	26,867	43.3
1920	129,614	40,688	45.8
1930	149,900	20,286	15.7

During the early 1890's improvements to increase the supply at Ludlow Reservoir were made in the form of diversion works. The Ludlow Supply formed the only source until 1908-09 when the Little River Supply Works were commenced and the Little River Intake Reservoir and Borden Brook Reservoir were constructed. These sources are located upon the opposite, or west, side of the City from the Ludlow Supply and required a new supply main. At this same time the distributive capacity of the system was increased by the installation of large diameter reinforcing mains in a circuit around the closely built-up section of the city.

From 1927 to 1932 the Cobble Mountain Reservoir Development,

Provin Mountain Reservoirs and additional transmission facilities were constructed, completing the main elements of the property as they now exist. The total storage, all served by gravity, consists of units as shown in table 2.

The water system of Springfield, as now developed, is the result of seventy-five years of growth. During this period the City has grown from approximately 17,000 to over 150,000 and between 1870 and 1920 the increase per decade varied between 25 and 46 per cent. From 1920 to 1930 the increase was approximately 16 per cent. This exceptionally large and rapid growth has required continuous enlargement and expansion of the system, both supply and distributive, with the consequent replacement of existing with new and larger facilities. The sources of supply have shifted in direction from one side of the city to the opposite side.

TABLE 2  
*Springfield, Mass., Storage Capacity*

	<i>million gallons</i>
<b>Westfield-Little River System</b>	
Borden Brook Impounding Reservoir.....	2,500
Cobble Mountain.....	22,225
Intake (diverting) Reservoir.....	41
Sedimentation Basin.....	39
Provin Mt. Reservoir No. 1 (Distributing).....	17
Provin Mt. Reservoir No. 2 (Distributing).....	12
Total Regular System.....	24,834
Ludlow Reservoir, emergency system.....	1,434
Grand Total.....	26,268

Originally served from reservoirs at low elevations, two zones of pressures resulted after the completion of the Ludlow Reservoir. Upon the retirement of the lower sources and the establishment of one service level, the low service mains were forced to stand a considerably higher pressure. The Little River Supply added another 12 to 15 pounds pressure. At the present time maximum pressures of 147 lb. exist in the main street levels, 87 lb. in the hill section and 81 lb. in Ludlow. Some difficulty was experienced when these changes were first made and some replacements were required because of lack of strength of pipe material.

Changes in street grades and other underground facilities due to

growth have likewise made necessary changes in distribution mains.

As a result of all of these various factors causing the retirement, replacement and extension of elements of the system including the meeting in full of the requirements of the National Board of Fire Underwriters, it is difficult to conceive of any water works in this country which has faced a more severe experience in the retirement of main pipe facilities than has the Springfield plant.

### **Basis of Study**

In a study of this kind, any attempt to draw conclusions as to the service lives of the various main pipe elements would be inaccurate without a definite knowledge of the dates at which they have been installed and the dates at which retirements have taken place. In Springfield accurate records are available in annual reports from 1864 to the present date. From 1864 to 1875 the records are, in some respects, difficult to follow because of changes in street names and lack of accurate designations by streets. However, the accuracy of the data regarding these early cement mains is not essentially affected and, in any event, there is not much value in determining the lives of these pipes except as a matter of historical and academic interest.

Following 1875, year by year, the additions and retirements have been set forth in the reports of the department, by street and sizes and classes of pipe. Changes of street names have been set forth enabling the trail of the pipe to be maintained. Where corrections to pipe previously installed or retired have been necessary in later years, such corrections are designated in the reports and in practically all cases can be traced. In 1907, all pipe previously installed was remeasured in the field and minor adjustments in lengths were made. These corrections have been applied to the date at which the pipe was originally laid.

In a few instances, pipe was purchased from other owners. Where the date of the original installation was determinable, these pipes were included, as were also small wrought-iron pipes laid by private parties. In a few instances the date of installation of purchased pipe could not be determined, in which case the pipe was not included in this study. The pipes used for hydrant laterals were eliminated inasmuch as they form no part of the main distributive element of the property, individually consist of many short lengths and in some years are not fully identified.

**TABLE 3**  
*Springfield, Mass. Total Footage of Pipe*  
 (Not including hydrant laterals, and unidentified pipe)

YEAR	PLACED	RETIRED	IN SERVICE	YEAR	PLACED	RETIRED	IN SERVICE
1864	8,602	Retirements of original wood pipe not shown	8,602	1908	1,051,325	210,373	840,952
1865	44,756		44,756	1909	1,151,770	246,033	905,737
1866	77,088		77,088	1910	1,201,337	256,449	944,888
1867	86,137		86,137	1911	1,243,098	266,611	976,487
1868	93,329		93,329	1912	1,282,476	273,406	1,009,070
1869	103,874		103,874	1913	1,348,070	290,189	1,057,881
1870	109,420		109,420	1914	1,412,930	317,226	1,095,704
1871	118,140		118,140	1915	1,455,122	329,117	1,126,005
1872	125,289		125,289	1916	1,481,634	331,106	1,150,528
1873	133,431		133,431	1917	1,505,445	335,986	1,169,459
1874	228,596		228,596	1918	1,512,746	338,901	1,173,845
1875	283,122		283,122	1919	1,532,037	339,630	1,192,407
1876	301,669		301,669	1920	1,562,842	343,978	1,218,864
1877	314,065		314,065	1921	1,604,566	350,002	1,254,564
1878	319,613		319,613	1922	1,644,458	351,873	1,292,585
1879	327,319		327,319	1923	1,683,508	354,429	1,329,079
1880	334,584		334,584	1924	1,743,254	358,563	1,384,691
1881	346,748		346,748	1925	1,799,476	363,149	1,436,327
1882	356,001		356,001	1926	1,840,490	370,738	1,469,752
1883	367,490		367,490	1927	1,863,163	375,076	1,488,087
1884	383,464	2,264	381,200	1928	1,884,763	379,494	1,505,269
1885	398,942	7,704	391,238	1929	1,960,535	387,237	1,573,298
1886	422,746	13,299	409,447	1930	1,984,333	394,349	1,589,984
1887	452,319	26,389	425,930	1931	2,000,842	401,274	1,599,568
1888	484,920	42,275	442,645	1932	2,022,885	408,785	1,614,100
1889	519,058	46,694	472,364	1933	2,038,439	415,675	1,622,764
1890	540,997	63,775	477,222	1934	2,048,680	423,501	1,625,179
1891	566,100	76,096	490,004	1935	2,060,372	435,215	1,625,157
1892	594,376	86,308	508,068	1936	2,081,480	448,582	1,632,898
1893	633,183	94,103	539,080	1937	2,101,953	451,814	1,650,139
1894	701,670	98,806	601,864	1938	2,120,639	458,143	1,662,496
1895	738,208	103,721	634,487	Total identified pipe,			
1896	768,321	112,188	656,133	1938.....			1,662,496
1897	813,778	120,397	693,381	Hydrant Laterals.....			24,475
1898	850,367	129,768	720,599	Purchased, Ludlow Mfg.			
1899	873,816	133,757	740,059	Associates, 1937.....			53,840
1900	885,367	136,938	748,429	Purchased former years,			
1901	895,729	139,322	756,407	unidentified.....			5,045
1902	900,950	139,662	761,288	Not identified.....			12,456
1903	918,217	140,488	777,729	Total pipe in place,			
1904	927,793	144,161	783,632	12/31/1938.....			1,758,312
1905	950,111	151,078	799,033				
1906	971,047	159,820	811,227				
1907	998,048	168,158	829,890				

Table 3 shows the growth of the pipe system, the total pipe installed, the amount retired and the footage of mains in service in each year from 1864 to 1938 with the above noted exceptions. The total pipe in service at the end of 1938 which could be identified was 1,662,496 feet. The total footage of pipe in service at the end of 1938, 1,758,312 feet, includes 24,475 feet of hydrant laterals, 58,885 feet of purchased pipe and 12,456 feet of pipe which could not be

TABLE 4

*Springfield, Mass. Footage of Pipe in Service Dec. 31, 1938*  
(Not including hydrant laterals or unidentified pipe of all kinds)

SIZE	WROUGHT IRON	CEMENT	CAST IRON	STEEL	TRANSITE	TOTAL
<i>inches</i>						
1	7,263					7,263
1½	23,283					23,283
1½	15,072					15,072
2	41,940					41,940
4			41,581			41,581
6			448,182			448,182
8			583,032	4	1,049	584,085
10			69,781	28	1,232	71,041
12			95,984			95,984
16			76,287	253		76,540
20			7,903	125		8,028
24		19,460	43,997	71		63,528
30			50,193	2,163		52,356
36			30,849	3,822		34,671
42			2,179	61,913		64,092
48				18,309		18,309
54				16,168		16,168
60				71		71
66				279		279
72				23		23
Total.....	87,558	19,460	1,449,968	103,229	2,281	1,662,496

accurately identified by year of installation. The growth of the pipe system is also illustrated in fig. 1 in relation to the growth in population.

Figure 2 shows the main portion of the distribution system in the City of Springfield.

Table 4 gives the total identified pipe by sizes and kinds in service at the end of 1938.

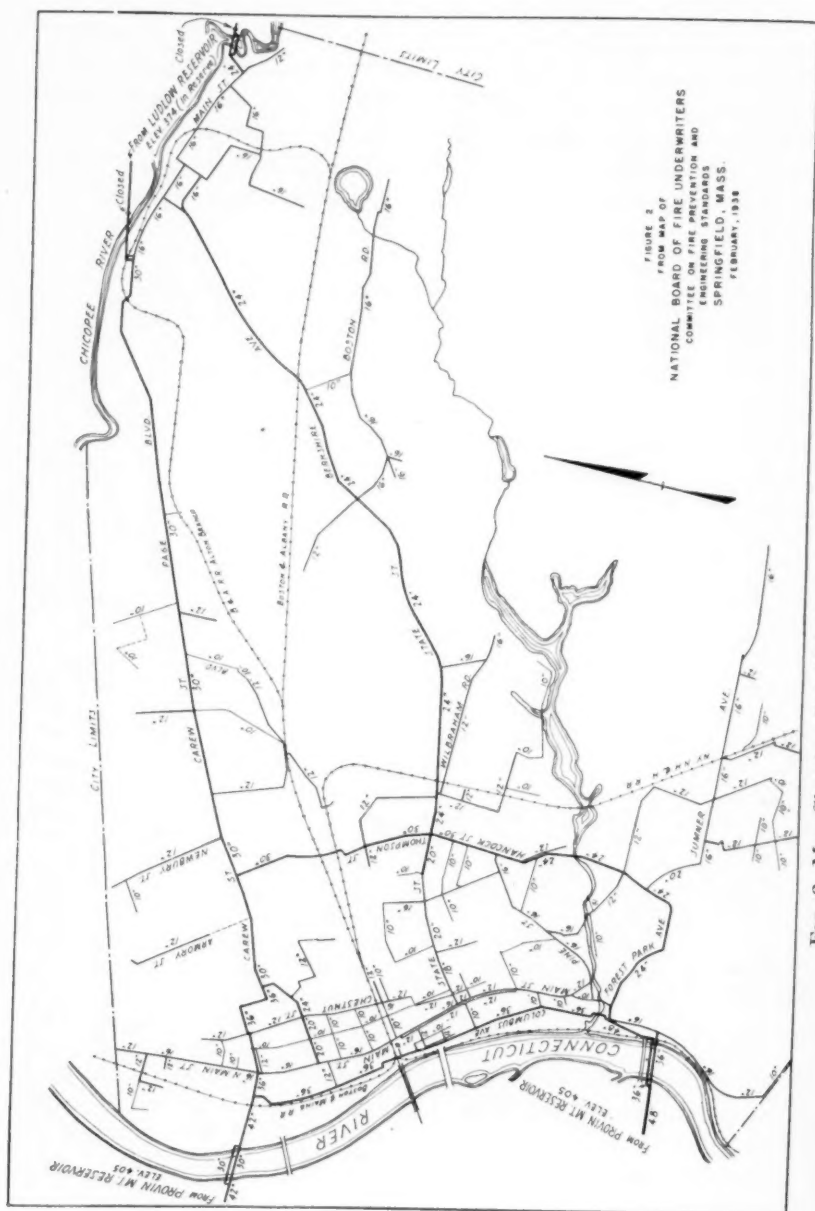


Fig. 2. Map Showing Chief Mains of Springfield, Mass. System

Table 5 indicates the retirement of cement pipe by sizes and years in which retired. Of a total of 250,961 feet of cement pipe installed, 231,501 feet have been retired leaving 19,460 feet still in service.

Table 6 shows the retirement of small wrought-iron pipe by sizes and years retired. Wrought-iron mains identified totaled 202,079 feet, of which 114,521 feet have been retired leaving, at the end of 1938, 87,558 feet still in service.

Table 7 indicates the similar data for the identified cast-iron mains. There have been a total of 1,564,370 feet of cast-iron pipe installed and 112,121 feet retired leaving the net total of 1,452,249 feet in service as of the end of 1938.

### Mortality Data

In determining the mortality rates of the various kinds and sizes of pipe, one-year intervals were used in the studies. In view of the long lives of the pipe of the various classes, particularly in the case of cast-iron pipe, and the advantages in securing sufficient retirements to develop the forms of the mortality survivor curves and, more important still, to include all of the factors, throughout the history of the plant, which affected the retirement of pipe, the entire period of service for which the data were available was used.

Factors producing retirement of cast-iron pipe are largely supersession because of inadequacy due to growth, changes in sources of supply or pumping methods, acts of public authority, other utilities or services causing interference with the pipe and changes wrought by extensions to the system. Actual depreciation of the pipe material, wear and tear, changes in the art of manufacture or changes in policy play very little part in the retirement of cast-iron pipe. Therefore the entire service record of such pipe can be used to indicate what may be expected in such long-lived facilities rather than to use only a three-, five- or ten-year period of recent retirement experience.

No attempt has been made herein to define the terms used in mortality studies since these are available in standard works of reference.

### Cement Mains

In tables 8, 9, and 10 are given the data from which the actual mortality survivor curves of fig. 3 for the 2-, 3- and 4-inch; 6-, 8- and 10-inch and the 12- to 24-inch cement-wrought-iron pipe are developed.

The results of the study of the mortality of these mains are of little

(Length in feet)											
YEAR	2"	3"	4"	6"	8"	10"	12"	16"	20"	24"	TOTAL
1884		1,935	329								2,264
1885		1,687	156	114	655		2,526				5,353
1886	230	2,671	167		1,796					215	4,864
1887	313	8,481	1,109	1,592	577						12,072
1888		12,236	433			1,075	1,193				14,937
1889		1,317	799	49	1,157						3,302
1890	386	4,395	496	6,893	2,530	1,410		318			16,428
1891		7,032	28	3,374	1,337						11,771
1892		2,647		90	1,555		3,462				7,754
1893		130		5,305							5,435
1894		550		460							1,010
1895		1,221		38							1,259
1896				1,597							1,597
1897		2,497	1,021	2,236	1,761		28				7,543
1898	361	3,161	1,595	3,051							8,168
1899		287	324	473							1,084
1900			1,204	1,481					265		2,950
1901	320				1,835						2,155
1902											—
1903			23								23
1904			1,266	1,857							3,123
1905		644		2,123							2,767
1906			459	1,452							1,911
1907			369		2,171		2,563				5,103
1908		456	4,530	13,287	7,991		3,563	4,066			34,007
1909		418	2,232	1,398	1,800		2,690	7,683	4,100	114	20,321
1910				28	118			4,139			4,285
1911			127	995	70						2,814
1912				1,373	94						2,531
1913			573	469	1,368		356				3,761
1914			1,094	1,416	3,289		700				13,989
1915			1,150	2,586	1,355						5,091
1936											9,905
Total placed	1,610	51,765	19,464	53,737	31,459	2,485	17,081	16,206	4,365	52,789	250,961
Total retired	1,610	51,765	19,464	53,737	31,459	2,485	17,081	16,206	4,365	33,329	231,501
Total in service	0	0	0	0	0	0	0	0	0	19,460	19,460

TABLE 6  
Springfield, Mass. Retirements of Wrought-Iron Mains  
(Length in feet)

YEAR	1'	1'	1½'	1½'	2'	3'	TOTAL
1886		255	476				731
1887				49	969		1,018
1888		100	211		85		396
1889			387		602		989
1890					399		399
1891					469		469
1892		447	331		1,664		2,442
1893			200	63			263
1894		250	1,363	273	1,398	1,063	4,347
1895		200	737		1,602		2,539
1896			31	82	2,967		3,080
1897		234	432				666
1898	180		94		894	11	1,179
1899		262	209	52	1,157		1,680
1900			66				66
1901					182		182
1902		260					260
1903		220					220
1904		10	321	36			367
1905		81	552		2,154		2,787
1906		68	527		2,984		3,579
1907			1,242				1,242
1908		121	755		2,401		3,277
1909		110	33			90	233
1910		100	805				905
1911		22	493	1,273			1,788
1912		4	338	673	724	39	1,778
1913		368	292	626	1,311	362	2,959
1914		352	544	102	404		1,402
1915		456	267		2,848	14	3,585
1916		174	53		503		730
1917		231	172	406			809
1918					180		180
1919		261		114	59		434
1920		58	310		549	437	1,354
1921		1,593		251	2,335		4,179
1922		503	744	290	60		1,597
1923	19	305	461		475		1,260
1924		463	1,984	648	1,039		4,134
1925		1,484	1,307	919	639		4,349
1926		1,496	1,386	1,861	2,117		6,860
1927		562	735	193	482		1,972
1928		321	1,077	450	500		2,783
1929	80	759	1,780	1,368	2,355		6,342
1930		36	1,056	365	2,811		4,268
1931		174	257	769	911		2,111
1932		912	1,353	2,655	2,109		7,029
1933		43	449	108	3,557		4,157
1934			706	67	2,214		2,987
1935					4,245		4,245
1936	195	119		76	1,369		1,759
1937		63	827	255	295		1,440
1938		280	1,651	385	2,398		4,714
Total placed	909	21,020	50,297	29,481	98,356	2,016	202,079
Total retired	909	13,757	27,014	14,409	56,416	2,016	114,521
Total in service	0	7,263	23,283	15,072	41,940	0	87,558

**TABLE 7**  
*Springfield, Mass. Retirements of Cast-Iron Mains*  
(Length in feet)

YEAR	3"	4"	6"	8"	10"	12"	16"	20"	24"	30"	36"	42"	TOTAL
1885		87											87
1888		525		28									553
1889		80	48										128
1890	34		220										254
1891			53		28								81
1892	16												16
1893			1117	980									2097
1894									346				346
1895		67		50									117
1896		455		3335									3790
1898		24											24
1899				1225									1225
1900			49	116									165
1901				47									47
1902		24		56									80
1903		362	221										583
1904		16	167										183
1905		415	538	410									1363
1906		2169	1083										3252
1907		317	628	892		156							1993
1908	153	1646	1456	1621		55							4931
1909	800	1034	7336	3484		1654	788	10					15106
1910		1018	2229	1040		939							5226
1911		873	3408	87									4368
1912		69	910	40									1019
1913		367	4563	1393	752	214	8						7297
1914		267	4056	506	16	7			135		160		5147
1915			1271	1883						61			3215
1916			1259										1259
1917		2046	447							1578			4071
1918		16	961	184	1574								2735
1919		148	147										295
1920		43	204				2747						2994
1921		720	962			163							1845
1922			274										274
1923		1273		23									1296
1925			237										237
1926			492	237									729
1927		718	1184	203		261							2366
1928		65	812	36		722							1635
1929			253		249	899							1401
1930		1329	816	184		515							2844
1931		4164	181	394								75	4814
1932			482										482
1933		716	1780	237									2733
1934		3165	1674										4839
1935		6953	516										7469
1936		1555	11	118	10				9				1703
1937		96	1133	407			44		112				1792
1938		587	239	750							39		1615
Total installed	100374990	491599604047*	73642*	10156979874791344599518323104822541,564,370									
Total retired	100333409	43417	19966	2629	5585	3587	10	602	1639	199	75		112,121
Total in service	041581448182584081*	71013*	9598476287790343997501933084921791,452,249										

\* Includes 1,049 ft. of 8" and 1,232 ft. of 10" transite pipe.

TABLE 8  
*Springfield, Mass. Percentage Survival 2-, 3-, and 4-inch Cement-Wrought-Iron Mains*

AGE INTERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIREMENT RATE	PER CENT OF SURVIVAL
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>
0-1	72,839	0	0	100.000
1-2	72,839	0	0	100.000
2-3	72,839	0	0	100.000
3-4	72,839	0	0	100.000
4-5	72,839	0	0	100.000
5-6	72,839	0	0	100.000
6-7	72,839	0	0	100.000
7-8	72,839	0	0	100.000
8-9	72,839	0	0	100.000
9-10	72,839	0	0	100.000
10-11	72,839	0	0	100.000
11-12	72,839	23	0.032	99.968
12-13	72,816	0	0	99.968
13-14	72,816	50	0.069	99.899
14-15	72,766	0	0	99.899
15-16	72,766	99	0.136	99.763
16-17	72,677	1,240	1.706	98.061
17-18	71,427	1,633	2.286	95.819
18-19	69,794	1,677	2.403	93.516
19-20	68,117	5,738	8.424	85.638
20-21	62,379	2,525	4.048	82.171
21-22	59,854	9,099	15.202	69.679
22-23	50,755	6,612	13.027	60.602
23-24	44,143	9,045	20.490	48.185
24-25	35,098	2,476	7.055	44.786
25-26	32,622	6,560	20.109	35.780
26-27	26,062	1,568	6.016	33.627
27-28	24,494	230	0.939	33.311
28-29	24,264	2,247	9.261	30.226
29-30	22,017	4,601	20.897	23.910
30-31	17,416	1,058	6.075	22.457
31-32	16,358	579	3.540	21.662
32-33	15,799	2,367	14.982	18.417
33-34	13,412	3,336	24.873	13.836
34-35	10,076	227	2.253	13.524
35-36	9,849	1,527	15.504	11.427
36-37	8,322	1,914	22.999	8.799
37-38	6,408	23	0.359	8.767
38-39	6,385	3,026	47.392	4.612
39-40	3,359	711	21.167	3.636
40-41	2,648	1,358	51.284	1.771
41-42	1,290	49	3.798	1.704
42-43	1,241	0	0	1.704
43-44	1,241	1,241	100.	0
44-45	0	0	0	0
	1,869,900	72,839	3.895	2567.124

Average life 25.67 years.

TABLE 9  
*Springfield, Mass. Percentage Survival 6-, 8-, and 10-inch Cement-Wrought-Iron Mains*

AGE INTERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIREMENT RATE	PER CENT OF SURVIVAL
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>
0-1	87,681	0	0	100.000
1-2	87,681	0	0	100.000
2-3	87,681	0	0	100.000
3-4	87,681	0	0	100.000
4-5	87,681	0	0	100.000
5-6	87,681	0	0	100.000
6-7	87,681	0	0	100.000
7-8	87,681	0	0	100.000
8-9	87,681	0	0	100.000
9-10	87,681	0	0	100.000
10-11	87,681	0	0	100.000
11-12	87,681	0	0	100.000
12-13	87,681	0	0	100.000
13-14	87,681	0	0	100.000
14-15	87,681	0	0	100.000
15-16	87,681	560	0.639	100.000
16-17	87,121	0	0	99.361
17-18	87,121	1,075	1.234	99.361
18-19	86,046	27	0.031	98.135
19-20	86,019	244	0.284	98.105
20-21	85,775	1,403	1.636	97.826
21-22	84,372	2,698	3.198	96.226
22-23	81,674	2,141	2.621	93.149
23-24	79,533	2,197	2.762	90.708
24-25	77,336	7,388	9.553	88.203
25-26	69,948	1,877	2.683	79.777
26-27	68,071	5,609	8.240	71.240
27-28	62,462	4,314	6.907	66.319
28-29	58,148	2,355	4.050	63.633
29-30	55,793	2,384	4.273	60.914
30-31	53,409	2,638	4.939	57.905
31-32	50,771	1,955	3.851	55.675
32-33	48,816	3,982	8.157	51.134
33-34	44,834	15,422	34.398	33.545
34-35	29,412	2,046	6.956	31.212
35-36	27,366	3,704	13.535	26.987
36-37	23,662	5,471	23.121	20.747
37-38	18,191	1,467	8.064	19.074
38-39	16,724	5,615	33.575	12.670
39-40	11,109	2,668	24.017	9.627
40-41	8,441	6,180	73.214	2.579
41-42	2,261	112	4.954	2.451
42-43	2,149	0	0	2.451
43-44	2,149	349	16.240	2.053
44-45	1,800	886	49.222	1.042
45-46	914	914	100.	0
46-47	0	0	0	0
	2,726,642	87,681	3.216	3109.746

Average life, 31.10 years.

other than academic interest at the present time as the use of this type of pipe is no longer common in this country. The average life of these pipe in Springfield is indicated in table 11.

It is interesting to note that 19,460 feet of 24-inch cement-wrought-iron pipe, laid in 1874, is still in service after 65 years. It was necessary to assume the retirement of this pipe in six years to determine the average lives of these pipe as herein estimated.

### Wrought-Iron Mains

Wrought-iron mains, in the experience of the Springfield system, have been installed to a large extent by private parties to secure water

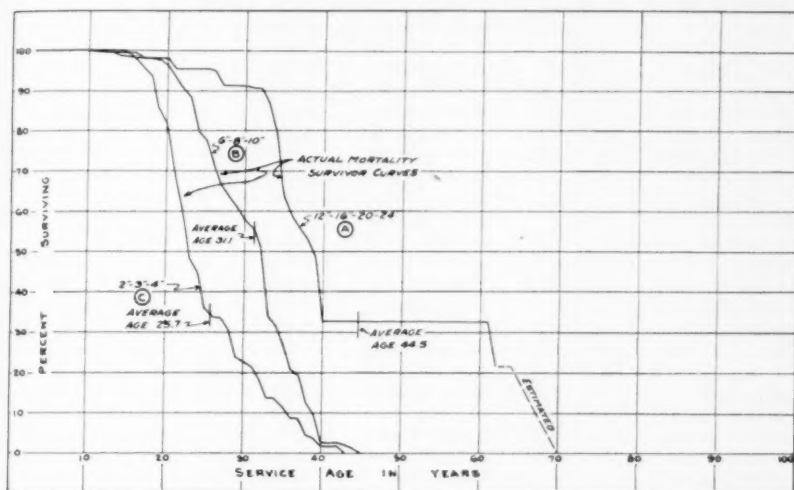


FIG. 3. Mortality Curves based on records of lives of cement pipe, Springfield, Mass.

service in individual cases in advance of the need for fire service or large mains. As a consequence a very large percentage of these mains, from  $\frac{3}{4}$  to 2 inches in diameter, was laid and replaced by larger mains within a few years thereafter. In many systems, however, even though replacements may take place at a relatively early period it would not happen at nearly so rapid a rate as was the experience in Springfield.

Tables 12 and 13 show the development of the mortality survival data shown on fig. 4. The average life of the  $\frac{3}{4}$ - to  $1\frac{1}{2}$ -inch wrought-iron mains was determined to be 18.9 years with an estimated end age of 50 years. For the 2-inch wrought-iron mains the average life was indicated as 22.8 years with an end age of 60 years.

TABLE 10  
Springfield, Mass. Percentage Survival 12-, 16-, 20-, 24-inch Cement-  
Wrought-Iron Mains

AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SUR- VIVAL	AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL
years	feet	feet	%	%	years	feet	feet	%	%
0-1	90,441	0	0	100.000	37-38	53,160	2,814	5.293	55.668
1-2	90,441	0	0	100.000	38-39	50,346	2,531	5.027	52.870
2-3	90,441	0	0	100.000	39-40	47,815	3,761	7.866	48.711
3-4	90,441	0	0	100.000	40-41	44,054	14,689	33.343	32.469
4-5	90,441	0	0	100.000	41-42	29,365	0	0	32.469
5-6	90,441	0	0	100.000	42-43	29,365	0	0	32.469
6-7	90,441	0	0	100.000	43-44	29,365	0	0	32.469
7-8	90,441	0	0	100.000	44-45	29,365	0	0	32.469
8-9	90,441	0	0	100.000	45-46	29,365	0	0	32.469
9-10	90,441	0	0	100.000	46-47	29,365	0	0	32.469
10-11	90,441	0	0	100.000	47-48	29,365	0	0	32.469
11-12	90,441	215	0.238	99.762	48-49	29,365	0	0	32.469
12-13	90,226	0	0	99.762	49-50	29,365	0	0	32.469
13-14	90,226	0	0	99.762	50-51	29,365	0	0	32.469
14-15	90,226	960	1.064	98.701	51-52	29,365	0	0	32.469
15-16	89,266	0	0	98.701	52-53	29,365	0	0	32.469
16-17	89,266	318	0.356	98.350	53-54	29,365	0	0	32.469
17-18	88,948	233	0.262	98.092	54-55	29,365	0	0	32.469
18-19	88,715	0	0	98.092	55-56	29,365	0	0	32.469
19-20	88,715	0	0	98.092	56-57	29,365	0	0	32.469
20-21	88,715	0	0	98.092	58-59	29,365	0	0	32.469
21-22	88,715	2,526	2.847	95.299	59-60	29,365	0	0	32.469
22-23	86,189	0	0	95.299	60-61	29,365	0	0	32.469
23-24	86,189	28	0.032	95.269	61-62	29,365	0	0	32.469
24-25	86,161	0	0	95.269	62-63	29,365	9,905	33.731	21.517
25-26	86,161	0	0	95.269	63-64	19,460	0	0	21.517
26-27	86,161	265	0.308	94.976	64-65	19,460	0	0	21.517
27-28	85,896	3,462	4.030	91.148					
28-29	82,434	0	0	91.148	65-66	19,460	3,360	17.266	17.802
29-30	82,434	0	0	91.148	66-67	16,100	3,300	20.497	14.143
30-31	82,434	84	0.102	91.055	67-68	12,800	3,200	25.000	10.607
31-32	82,350	356	0.432	90.662	68-69	9,600	3,200	33.333	7.071
32-33	81,994	0	0	90.662	69-70	6,400	3,200	50.000	3.535
33-34	81,994	3,813	4.650	86.446	70-71	3,200	3,200	100.000	0
34-35	78,181	6,819	8.722	78.906	71-72	0	0	0	0
35-36	71,362	14,063	19.707	63.356					
36-37	57,299	4,139	7.224	58.779		4,023,628	90,441	2.248	4,448.904

Average life, 44.49 years.

Note: Figures below dotted line, estimated.

TABLE 11  
Average Life of Cement-Wrought-Iron Pipe

SIZES	AVERAGE LIFE	END AGE
	years	years
2-, 3- and 4-inch.....	25.7	44
6-, 8- and 10-inch.....	31.1	46
12-, 16-, 20- and 24-inch.....	Est. 44.5	Est. 71

TABLE 12  
Springfield, Mass. Percentage Survival  $\frac{3}{4}$ -, 1-,  $1\frac{1}{4}$ -, and  $1\frac{1}{2}$ -inch Wrought-Iron  
Mains

AGE INTERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIREMENT RATE	PER CENT OF SURVIVAL
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>
0-1	101,697	552	0.543	99.457
1-2	100,726	4,745	4.711	94.772
2-3	93,650	3,734	3.987	90.993
3-4	86,898	2,327	2.678	88.556
4-5	84,571	3,532	4.176	84.858
5-6	80,067	2,606	3.255	82.096
6-7	76,606	2,617	3.416	79.292
7-8	72,708	4,582	6.302	74.295
8-9	67,557	2,325	3.442	71.738
9-10	62,473	2,052	3.285	69.381
10-11	59,672	3,268	5.477	65.581
11-12	55,202	2,340	4.239	62.801
12-13	51,302	1,301	2.536	61.208
13-14	47,018	1,110	2.361	59.763
14-15	40,989	2,072	5.055	56.742
15-16	33,446	1,481	4.428	54.229
16-17	28,700	2,556	8.906	49.399
17-18	22,202	2,283	10.283	44.319
18-19	19,165	880	4.592	42.284
19-20	16,713	433	2.591	41.188
20-21	15,606	277	1.775	40.457
21-22	14,944	1,257	8.411	37.054
22-23	13,523	1,540	11.388	32.834
23-24	11,561	131	1.133	32.462
24-25	11,128	0	0	32.462
25-26	10,245	632	6.169	30.459
26-27	9,604	279	2.905	29.574
27-28	8,940	386	4.318	28.297
28-29	7,841	1,578	20.125	22.602
29-30	6,263	368	5.876	21.274
30-31	5,670	163	2.875	20.662
31-32	5,507	173	3.141	20.013
32-33	5,334	0	0	20.013
33-34	4,733	680	14.367	17.138
34-35	4,053	415	10.239	15.383
35-36	3,439	251	7.299	14.260
36-37	3,055	0	0	14.260
37-38	2,946	487	16.531	11.903
38-39	2,268	324	14.286	10.203
39-40	1,924	0	0	10.203
40-41	1,388	0	0	10.203
41-42	936	0	0	10.203
42-43	770	0	0	10.203
43-44	770	0	0	10.203
44-45	770	0	0	10.203
45-46	406	352	86.700	1.357
46-47	54	0	0	1.357
47-48	54	0	0	1.357
48-49	54	27	50.000	.678
49-50	27	27	100.000	0
50-51	0	0	0	0
	1,253,478	56,143	4.479	1890.229

Average life, 18.90 years. Average age of survivors, 13.59 years.  
Note: Figures below dotted line, estimated.

### Cast-Iron Mains

After a preliminary study it was determined to group the mortality data for the cast-iron mains under 4-, 6-, 8-, and 10-inch and over diameters. The shape of the mortality curve for the 4-inch mains is very different from those of the other sizes. Little difference exists, however, in the mortality survival curves of the 6-inch and larger mains as they are thus far developed.

TABLE 13  
*Springfield, Mass. Percentage Survival 2-inch Wrought-Iron Mains*

AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SUR- VIVAL	AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL
years	feet	feet	%	%	years	feet	feet	%	%
0-1	100,372	253	0.252	99.748	31-32	11,974	280	2.338	28.891
1-2	99,118	2,442	2.464	97.290	32-33	11,694	1,131	9.672	26.097
2-3	95,020	2,984	3.140	94.235	33-34	10,518	45	0.428	25.985
3-4	91,504	2,830	3.093	91.319	34-35	10,473	526	5.022	24.680
4-5	88,668	594	0.670	90.707	35-36	9,931	1,242	12.506	21.594
5-6	87,762	1,849	2.107	88.796	36-37	8,662	1,136	13.115	18.762
6-7	85,654	268	0.313	88.518	37-38	7,059	328	4.647	17.890
7-8	85,386	1,552	1.818	86.909	38-39	6,731	898	13.341	15.503
8-9	83,029	883	1.063	85.985	39-40	5,731	515	8.986	14.110
9-10	81,621	3,409	4.177	82.393	40-41	5,155	404	7.837	13.004
10-11	76,787	3,849	5.013	78.263	41-42	4,063	331	8.147	11.945
11-12	70,966	1,880	2.649	76.190	42-43	3,732	84	2.251	11.676
12-13	65,530	1,247	1.903	74.740	43-44	3,648	0	0	11.676
13-14	62,665	2,678	4.274	71.546	44-45	2,216	0	0	11.676
14-15	55,152	3,195	5.793	67.401	45-46	2,058	0	0	11.676
15-16	45,327	2,316	5.110	63.957	46-47	2,058	93	4.519	11.148
16-17	39,782	1,231	3.094	61.978	47-48	1,753	0	0	11.148
17-18	36,381	2,023	5.561	58.531	48-49	1,591	0	0	11.148
18-19	29,973	3,588	11.971	51.524	49-50	1,591	222	13.953	9.593
19-20	26,248	4,332	16.504	43.020	50-51	1,369	410	29.949	6.720
20-21	21,916	845	3.856	41.361	51-52	959	0	0	6.720
21-22	21,071	1,532	7.271	38.354	52-53	959	190	19.812	5.389
22-23	19,391	30	0.155	38.295	53-54	769	488	63.459	1.969
23-24	18,608	236	1.268	37.509	54-55	281	0	0	1.969
24-25	18,201	90	0.494	37.622	55-56	281	0	0	1.969
25-26	18,111	1,009	5.571	35.526	56-57	281	0	0	1.969
26-27	17,102	1,358	7.941	32.705	57-58	281	0	0	1.969
27-28	14,620	299	2.045	32.036	58-59	281	0	0	1.969
28-29	13,973	295	2.111	31.360	59-60	281	281	100.000	0
29-30	12,946	366	2.827	30.473	60-61	0	0		0
30-31	12,499	365	2.920	29.583					
						1,611,391	58,432*	3.626	2,277.019

Average life, 22.77 years.

Average age of survivors, 15.53 years.

\* Includes 2016 feet of 3" pipe.

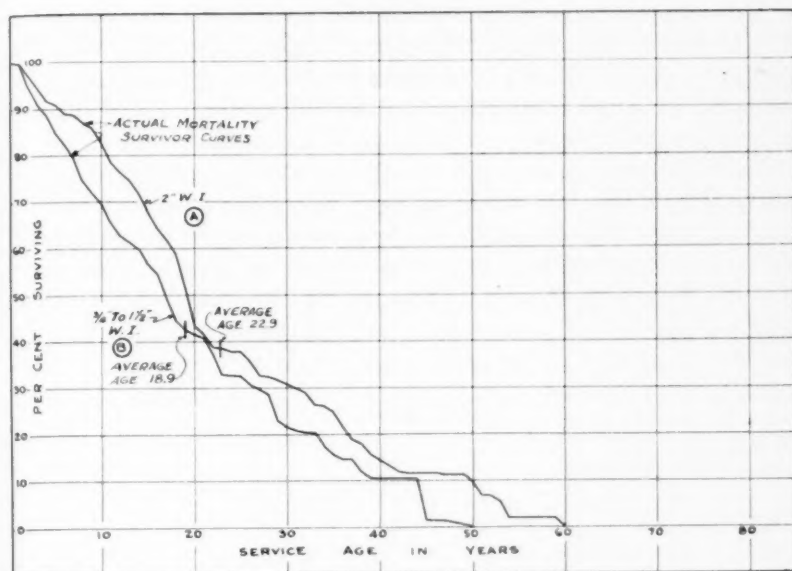


FIG. 4. Mortality Curves based on records of lives of wrought-iron pipe, Springfield, Mass.

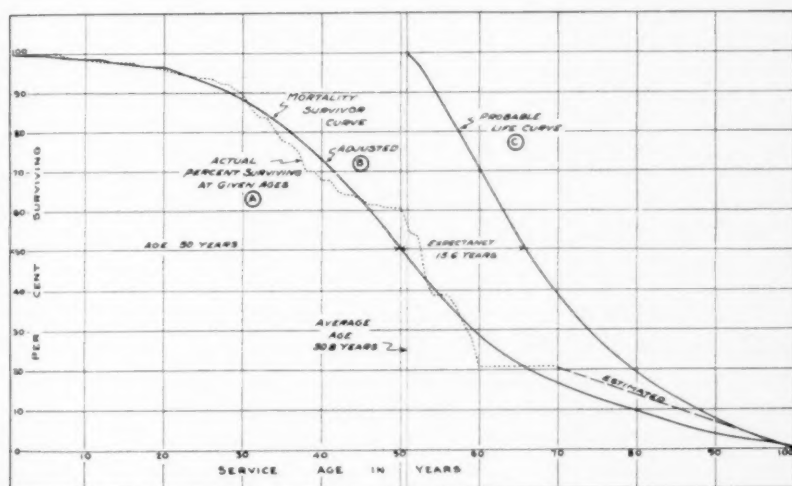


FIG. 5. Mortality Curves based on records of lives of 4-inch cast-iron pipe, Springfield, Mass.

Table 14 shows the computations to determine the survival ratios for the 4-inch cast-iron pipe, the mortality survival curve of which is shown on fig. 5. The computations for the 4-inch pipe have been adjusted to secure a smooth curve in order to formulate a life expectancy curve for the survivor units, which is also shown on fig. 5.

In the 69 years of experience of the 4-inch pipe the percentage of survival as shown by the actual retirements has reached 20.7. It was necessary to estimate the small remaining part of the completed curve in order to compute an average life and an end age. The latter was taken as 101 years and the average life determined at 50.8 years. The average annual rate of retirement of the 4-inch cast-iron pipe for the 69 years of experience has been 1.151 per cent.

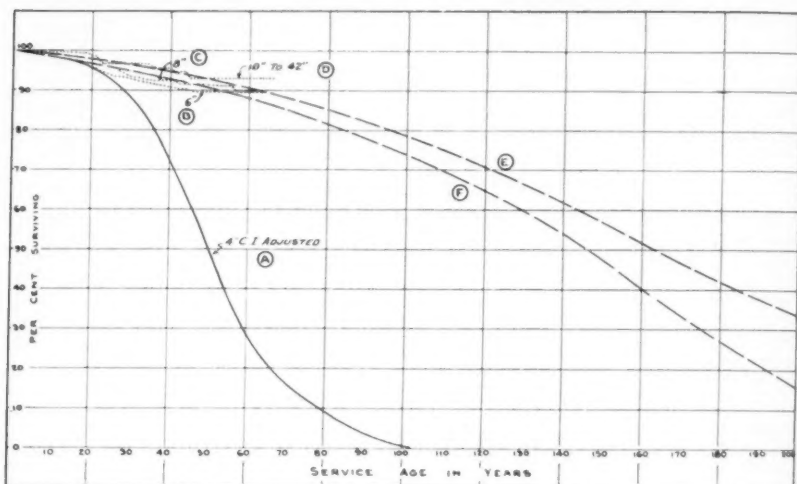


FIG. 6. Mortality Curves based on records of lives of cast-iron pipe, Springfield, Mass.

The mortality survival computations for the 6-, 8-, and 10-inch and larger diameter pipe are shown in tables 15, 16 and 17 upon which are based the curves shown on fig. 6. The adjusted mortality survival curve for the 4-inch cast-iron pipe is also shown on fig. 6.

The present mortality survival percentages and the average annual rates of retirements for the respective years of records of the various sizes of cast-iron pipe are shown in table 18.

### Conclusions

It is obvious if curves of the character of mortality survivor curves are to be drawn fairly to include the 64 years experience with 6- and

TABLE 14

*Springfield, Mass. Percentage Survival and Probable Life 4-inch Cast-Iron Mains*

AGE IN- TERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL	ADJUSTED CURVE	REMAIN- ING SERVICE	EXPECT- ANCY	PROBABLE LIFE
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>	<i>%</i>	<i>years</i>	<i>years</i>	<i>years</i>
0-1	74,990	9	.012	99.988	100.0	5080.4	50.84	50.84
1-2	74,981	14	.019	99.969	99.8	4980.4	49.90	50.90
2-3	74,967	31	.041	99.928	99.6	4880.6	49.00	51.00
3-4	74,705	12	.016	99.912	99.4	4781.0	48.10	51.10
4-5	74,683	0	.000	99.912	99.2	4681.6	47.19	51.19
5-6	74,683	0	.000	99.912	99.0	4582.4	46.29	51.29
6-7	74,405	546	.734	99.179	98.8	4483.4	45.39	51.39
7-8	73,359	289	.394	98.788	98.6	4384.6	44.47	51.47
8-9	73,070	43	.059	98.730	98.4	4286.0	43.56	51.56
9-10	73,027	37	.051	98.680	98.2	4187.6	42.64	51.64
10-11	72,493	157	.217	98.466	98.0	4089.4	41.73	51.73
11-12	71,785	6	.008	98.458	97.7	3991.4	40.85	51.85
12-13	71,779	565	.787	97.683	97.5	3893.7	39.94	51.94
13-14	71,214	19	.027	97.657	97.3	3796.2	39.02	52.02
14-15	71,045	42	.059	97.599	97.1	3698.9	38.09	52.09
15-16	71,003	197	.277	97.329	96.9	3601.8	37.17	52.17
16-17	70,524	421	.597	96.748	96.7	3504.9	36.25	52.25
17-18	70,091	228	.325	96.434	96.4	3408.2	35.35	52.35
18-19	69,735	129	.185	96.256	96.1	3311.8	34.46	52.46
19-20	69,310	35	.050	96.208	95.7	3215.7	33.60	52.60
20-21	69,140	314	.454	95.771	95.2	3120.0	32.77	52.77
21-22	68,826	1,004	1.459	94.374	94.6	3024.8	31.97	52.97
22-23	67,246	188	.280	94.110	94.1	2930.2	31.14	53.14
23-24	66,277	210	.317	93.812	93.4	2836.1	30.37	53.37
24-25	63,567	122	.192	93.632	92.6	2742.7	29.62	53.62
25-26	59,676	62	.104	93.535	91.8	2650.1	28.87	53.87
26-27	55,933	838	1.498	92.134	91.1	2558.3	28.08	54.08
27-28	53,457	47	.088	92.053	90.2	2467.2	27.35	54.35
28-29	51,021	724	1.419	90.747	89.2	2377.0	26.65	54.65
29-30	47,136	530	1.124	89.727	88.1	2287.8	25.97	54.97
30-31	45,220	1,928	4.264	85.901	87.1	2199.7	25.25	55.25
31-32	43,292	1,000	2.310	83.917	86.1	2112.6	24.54	55.54
32-33	42,254	160	.379	83.599	84.8	2026.5	23.90	55.90
33-34	42,086	1,627	3.866	80.367	83.3	1941.7	23.31	56.31
34-35	40,423	1,253	3.100	77.876	81.4	1858.4	22.83	56.83
35-36	39,150	502	1.282	76.878	80.1	1777.0	22.18	57.18
36-37	38,058	956	2.512	74.947	78.3	1696.9	21.67	57.67
37-38	37,102	778	6.733	69.901	76.6	1618.6	21.13	58.13
38-39	36,312	89	.245	69.730	74.8	1542.2	20.62	58.62
39-40	36,223	1,113	3.073	67.587	72.9	1467.2	20.13	59.13
40-41	33,736	76	.225	67.435	71.1	1394.3	19.61	59.61

TABLE 14—Continued

AGE IN- TERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL	ADJUSTED CURVE	REMAIN- ING SERVICE	EXPECT- ANCY	PROBABLE LIFE
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>	<i>%</i>	<i>years</i>	<i>years</i>	<i>years</i>
41-42	33,407	1,358	4.065	64.694	69.0	1323.2	19.18	60.18
42-43	30,803	410	1.331	63.833	66.4	1254.2	18.89	60.89
43-44	30,181	77	.255	63.670	64.4	1187.8	18.44	61.44
44-45	29,139	601	2.063	62.356	62.2	1123.4	18.06	62.06
45-46	27,842	520	1.868	61.191	60.2	1061.2	17.63	62.63
46-47	26,715	0	.000	61.191	57.9	1001.0	17.29	63.29
47-48	26,511	369	1.392	60.339	55.3	943.1	17.05	64.05
48-49	24,963	0	.000	60.339	52.6	887.8	16.88	64.88
49-50	24,488	50	.204	60.216	50.2	835.2	16.64	65.64
50-51	24,083	2,404	9.982	54.205	48.1	785.0	16.32	66.32
51-52	20,985	357	1.701	53.283	45.4	736.9	16.23	67.23
52-53	19,657	3,506	17.836	43.779	42.6	691.5	16.23	68.23
53-54	15,806	1,807	11.432	38.774	40.2	648.9	16.14	69.14
54-55	13,646	0	.000	38.774	38.1	608.7	15.98	69.98
55-56	13,646	0	.000	38.774	36.4	570.6	15.68	70.68
56-57	13,247	1,048	7.911	35.707	34.2	534.2	15.62	71.62
57-58	11,281	1,450	12.853	31.118	32.2	500.0	15.53	72.53
58-59	9,823	1,065	10.842	27.744	30.2	467.8	15.49	73.49
59-60	8,224	2,086	25.365	20.707	28.4	437.6	15.41	74.41
60-61	4,219	0	.000	20.707	26.9	409.2	15.21	75.21
61-62	3,234	0	.000	20.707	25.2	382.3	15.17	76.17
62-63	1,116	0	.000	20.707	23.6	357.1	15.13	77.13
63-64	173	0	.000	20.707	22.4	333.5	14.89	77.89
64-65	173	0	.000	20.707	21.1	311.1	14.74	78.74
65-66	173	0	.000	20.707	20.1	290.0	14.43	79.43
66-67	173	0	.000	20.707	19.1	269.9	14.13	80.13
67-68	173	0	.000	20.707	18.2	250.8	13.78	80.78
68-69	173	0	.000	20.707	17.2	232.6	13.52	81.52
TOTALS	2,902,038	33,409	1.151	4862.874				
69-70					16.6	215.4	12.98	81.98
70-71					15.7	198.8	12.66	82.66
71-72					14.7	183.1	12.46	83.46
72-73					13.8	168.4	12.20	84.20
73-74					12.9	154.6	11.98	84.98
74-75					12.3	141.7	11.52	85.52
75-76					11.6	129.4	11.16	86.16
76-77					11.0	117.8	10.71	86.71
77-78					10.5	106.8	10.17	87.17
78-79					9.9	96.3	9.73	87.73
79-80					9.3	86.4	9.29	88.29
80-81					8.7	77.1	8.86	88.86
81-82					8.1	68.4	8.44	89.44

TABLE 14—*Concluded*

AGE IN- TERVAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL	ADJUSTED CURVE	REMAIN- ING SERVICE	EXPECT- ANCY	PROBABLE LIFE
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>	<i>%</i>	<i>years</i>	<i>years</i>	<i>years</i>
82-83					7.5	60.3	8.04	90.04
83-84					6.8	52.8	7.76	90.76
84-85					6.3	46.0	7.30	91.30
85-86					5.7	39.7	6.96	91.96
86-87					5.1	34.0	6.67	92.67
87-88					4.5	28.9	6.42	93.42
88-89					4.0	24.4	6.10	94.10
89-90					3.6	20.4	5.67	94.67
90-91					3.2	16.8	5.25	95.25
91-92					2.8	13.6	4.86	95.86
92-93					2.5	10.8	4.32	96.32
93-94					2.2	8.3	3.77	96.77
94-95					1.9	6.1	3.21	97.21
95-96					1.6	4.2	2.63	97.63
96-97					1.4	2.6	1.86	97.86
97-98					1.0	1.2	1.20	98.20
98-99					0.8	.2	.25	98.25
99-100					0.4	.1	.25	99.25
100-101					0.1	.1	.10	100.10
					5080.4			

Estimated average life, 50.80 years.

Average age of survivors, 35.74 years.

Note: Figures below dotted line, estimated.

8-inch pipe in the Springfield system that they will assume approximately the position of the projected curves on fig. 6. Such curves indicate that the average life of 6-inch pipe will be somewhere in the neighborhood of 140 to 150 years and for 8-inch, 160 to 170 years. The curve for 10-inch and larger pipe would be farther to the right with a still longer average life. The age of the longest surviving units would approximate twice the average life and their retirement would take place in the dimly distant future.

In view of the common use of linings for pipes to carry the more corrosive waters and the progress that is being made in the treatment of water to inhibit corrosion, it appears reasonable to conclude that in the future the service lives of cast-iron pipe will universally tend to approach those resulting from necessary replacements due to changes in demand conditions. The experience in Springfield, where such

changes have been as numerous as can reasonably be expected because of the things that have happened in the development of that system, appears to indicate that our ideas of the length of service life of cast-iron pipe are due for revision upward. Furthermore, it must be borne in mind that growth in population as a cause of retirements

TABLE 15  
*Springfield, Mass. Percentage Survival 6-inch Cast-Iron Mains*

AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SUR- VIVAL	AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATE	PER CENT OF SURVIVAL
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>	<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>
0-1	491,599	9	.002	99.998	33-34	321,704	475	.148	91.898
1-2	488,668	37	.008	99.990	34-35	310,158	211	.068	91.836
2-3	486,065	261	.054	99.936	35-36	302,703	240	.079	91.763
3-4	485,020	151	.031	99.905	36-37	291,517	1,634	.561	91.248
4-5	484,310	949	.196	99.709	37-38	285,834	427	.149	91.112
5-6	482,889	678	.140	99.569	38-39	279,840	418	.149	90.976
6-7	482,098	414	.086	99.483	39-40	273,467	621	.227	90.769
7-8	479,920	310	.065	99.418	40-41	259,241	98	.038	90.735
8-9	479,450	660	.138	99.281	41-42	236,081	1,555	.659	90.137
9-10	476,967	1,244	.261	99.022	42-43	202,130	27	.013	90.125
10-11	474,797	588	.124	98.899	43-44	187,875	12	.006	90.120
11-12	472,727	3,079	.651	98.255	44-45	171,271	525	.307	89.843
12-13	468,204	1,459	.312	97.948	45-46	155,328	551	.355	89.524
13-14	466,208	396	.085	97.865	46-47	139,937	0	.000	89.524
14-15	464,244	1,037	.223	97.647	47-48	119,912	116	.097	89.437
15-16	461,269	2,208	.479	97.179	48-49	103,317	0	.000	89.437
16-17	458,886	1,204	.262	96.924	49-50	91,475	0	.000	89.437
17-18	456,993	1,039	.227	96.704	50-51	72,610	0	.000	89.437
18-19	454,741	1,983	.436	96.282	51-52	55,886	0	.000	89.437
19-20	452,445	2,544	.562	95.741	52-53	38,190	0	.000	89.437
20-21	449,435	685	.152	95.595	53-54	27,229	60	.220	89.240
21-22	448,459	4,818	1.074	94.568	54-55	22,623	0	.000	89.240
22-23	442,459	810	.183	94.395	55-56	14,589	0	.000	89.240
23-24	439,149	1,468	.334	94.080	56-57	11,942	0	.000	89.240
24-25	434,209	1,645	.379	93.723	57-58	11,852	0	.000	89.240
25-26	428,110	623	.146	93.586	58-59	4,905	0	.000	89.240
26-27	419,575	1,300	.310	93.296	59-60	4,905	0	.000	89.240
27-28	403,951	605	.150	93.156	60-61	4,905	0	.000	89.240
28-29	383,706	233	.061	93.099	61-62	4,905	0	.000	89.240
29-30	370,130	1,070	.289	92.830	62-63	2,813	0	.000	89.240
30-31	355,789	123	.035	92.798	63-64	1,231	0	.000	89.240
31-32	343,490	2,224	.647	92.198					
32-33	333,040	593	.178	92.034		18,729,477	43,417	0.232	5,972.922

Average age of survivors, 38.61 years.

will, according to present indications, become less in the future. Rates of growth of the communities served by this plant are decreasing as is the case in most cities. In fact, it is estimated by competent authorities that the population of the country as a whole will become stationary before the end of the century.

A number of interesting questions are raised by consideration of the facts found in this study and among them are these:

Is it correct to accrue depreciation on cast-iron pipe in the amount required for its complete replacement in the number of years represented by the average life expectancy when it is found that survivors

TABLE 16  
*Springfield, Mass. Percentage Survival 8-inch Cast-Iron Mains*

AGE INTER-VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE-MENT RATIO	PER CENT OF SUR-VIVAL	AGE INTER-VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE-MENT RATIO	PER CENT OF SURVIVAL
<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>	<i>years</i>	<i>feet</i>	<i>feet</i>	<i>%</i>	<i>%</i>
0-1	604,047	33	.005	99.995	33-34	95,020	0	.000	92.859
1-2	590,973	290	.049	99.946	34-35	88,161	189	.214	92.660
2-3	580,667	390	.067	99.879	35-36	87,274	128	.147	92.524
3-4	577,066	1,462	.253	99.626	36-37	84,516	0	.000	92.524
4-5	566,006	328	.058	99.568	37-38	84,516	0	.000	92.524
5-6	558,891	367	.066	99.502	38-39	83,441	21	.025	92.501
6-7	548,613	588	.107	99.396	39-40	80,963	63	.078	92.429
7-8	534,981	9	.002	99.394	40-41	77,805	7	.009	92.421
8-9	520,799	1,981	.380	99.016	41-42	73,988	686	.927	91.564
9-10	504,063	0	.000	99.016	42-43	68,245	0	.000	91.564
10-11	471,329	68	.014	99.002	43-44	59,017	136	.230	91.353
11-12	458,974	413	.090	98.913	44-45	51,743	0	.000	91.353
12-13	446,456	111	.025	98.888	45-46	49,707	37	.074	91.285
13-14	417,162	1,886	.452	98.441	46-47	47,257	0	.000	91.285
14-15	379,582	36	.009	98.432	47-48	45,043	0	.000	91.285
15-16	346,772	0	.000	98.432	48-49	40,821	0	.000	91.285
16-17	324,818	665	.205	98.230	49-50	40,084	115	.287	91.023
17-18	307,538	720	.234	98.000	50-51	38,622	0	.000	91.023
18-19	287,984	19	.007	97.993	51-52	34,262	0	.000	91.023
19-20	271,436	80	.029	97.965	52-53	29,791	0	.000	91.023
20-21	257,212	299	.116	97.851	53-54	26,527	0	.000	91.023
21-22	254,792	1,861	.730	97.137	54-55	26,017	0	.000	91.023
22-23	240,574	1,139	.473	96.678	55-56	24,402	0	.000	91.023
23-24	222,054	168	.076	96.605	56-57	24,102	327	1.357	89.788
24-25	192,443	12	.006	96.599	57-58	19,636	0	.000	89.788
25-26	170,754	93	.054	96.547	58-59	17,673	0	.000	89.788
26-27	148,103	2,998	2.024	94.593	59-60	14,271	0	.000	89.788
27-28	134,116	464	.346	94.266	60-61	14,271	0	.000	89.788
28-29	127,424	517	.406	93.883	61-62	14,271	0	.000	89.788
29-30	121,700	875	.719	93.208	62-63	14,271	0	.000	89.788
30-31	115,282	97	.084	93.130	63-64	7,489	0	.000	89.788
31-32	103,835	0	.000	93.130					
32-33	98,985	288	.291	92.859		12,948,637	19,966	0.154	6,043.011

Average age of survivors, 20.49 years.

beyond the average life continue in service for such a great number of years?

In such long-lived elements of property would it be fairer to the rate payers of all periods as well as to the owners of the property to provide a reserve for replacements slightly larger than the history of

TABLE 17  
Springfield, Mass. Percentage Survival 10-inch and Larger Cast Iron Mains

AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATIO	PER CENT OF SUR- VIVAL	AGE INTER- VAL	TOTAL IN SERVICE	TOTAL RETIRED	RETIRE- MENT RATIO	PER CENT OF SURVIVAL
years	feet	feet	%	%	years	feet	feet	%	%
0-1	392,731	0	.000	100.000	34-35	114,665	735	.641	96.053
1-2	391,428	0	.000	100.000	35-36	113,930	930	.816	95.269
2-3	387,953	10	.003	99.997	36-37	113,000	0	.000	95.269
3-4	374,918	8	.002	99.995	37-38	113,000	0	.000	95.269
4-5	373,391	0	.000	99.995	38-39	113,000	21	.019	95.251
5-6	371,733	167	.045	99.950	39-40	112,979	0	.000	95.251
6-7	367,461	6	.002	99.948	40-41	112,679	660	.586	94.693
7-8	362,373	0	.000	99.948	41-42	112,019	0	.000	94.693
8-9	361,700	752	.208	99.740	42-43	112,019	0	.000	94.693
9-10	357,363	265	.074	99.666	43-44	108,768	0	.000	94.693
10-11	356,445	0	.000	99.666	44-45	102,604	1,945	1.896	92.898
11-12	353,162	0	.000	99.666	45-46	54,244	0	.000	92.898
12-13	350,912	33	.009	99.657	46-47	43,058	0	.000	92.898
13-14	345,697	6	.002	99.655	47-48	41,866	0	.000	92.898
14-15	341,840	171	.050	99.605	48-49	41,866	0	.000	92.898
15-16	334,361	722	.216	99.390	49-50	38,803	0	.000	92.898
16-17	331,016	239	.072	99.318	50-51	34,263	0	.000	92.898
17-18	321,253	543	.169	99.150	51-52	31,609	0	.000	92.898
18-19	309,675	15	.005	99.145	52-53	31,609	0	.000	92.898
19-20	299,823	1,694	.565	98.585	53-54	27,989	0	.000	92.898
20-21	296,453	346	.117	98.470	54-55	21,805	0	.000	92.898
21-22	292,319	2,766	.946	97.538	55-56	19,481	0	.000	92.898
22-23	283,191	136	.048	97.491	56-57	16,793	0	.000	92.898
23-24	281,210	1,622	.577	96.928	57-58	15,588	0	.000	92.898
24-25	274,061	9	.003	96.925	58-59	15,588	0	.000	92.898
25-26	240,876	326	.135	96.794	59-60	15,050	0	.000	92.898
26-27	213,040	0	.000	96.794	60-61	15,019	0	.000	92.898
27-28	204,839	0	.000	96.794	61-62	15,019	0	.000	92.898
28-29	195,828	0	.000	96.794	62-63	15,019	0	.000	92.898
29-30	170,499	135	.079	96.718	63-64	15,019	0	.000	92.898
30-31	158,363	39	.025	96.694	64-65	8,192	0	.000	92.898
31-32	130,859	0	.000	96.694	65-66	1,088	0	.000	92.898
32-33	118,621	0	.000	96.694					
33-34	114,690	25	.022	96.673		11,807,715	14,326	0.121	6,345.897

Average age of survivors, 29.22 years.

TABLE 18  
Mortality Survival Percentages and Average Annual Retirement Rates  
of Cast-Iron Pipe

SIZE	YEARS OF RECORD	MORTALITY SUR- VIVAL PERCENTAGE	AVERAGE ANNUAL RETIREMENT PERCENTAGE
6-inch.....	64	89.24	0.232
8-inch.....	64	89.79	0.154
10-inch and larger.....	66	92.89	0.121

retirements shows is necessary, rather than one based on assumed average lives?

Is not the present activity on the part of accountants to urge depreciation provisions computed on estimated average life expectancy in order to simplify accounting procedure, when applied to long lived property, tending to get farther away from a proper charge to rate payers for depreciation rather than closer to it?

Is not there reason to believe that presently popular methods and assumptions as to service lives in accruing depreciation, when applied to water systems made up principally of long-lived cast-iron mains, will result in the accumulation of huge reserves decades before they will be needed?

Further study by the authors of the experience of the Springfield system may well form the basis of later comments. It is to be hoped that other cases will be found and investigated where complete long time records are available so that additional light will be thrown upon the important subject of the service lives of cast-iron mains.

**Discussion by Elson T. Killam.\*** The paper by Messrs. Newsom and Aldrich presents an unusual and valuable record of service life of water mains, and the importance of these data, and the interesting method of presentation, is such that it is to be hoped that in future years similar data will be made available for comparison with other systems.

The service life or the capacity life of pipe varies so greatly from structural life that the question is recognized to be of major importance. Except in unusual soil conditions the structural life ordinarily far exceeds the normal bond period of 30 years. On the other hand, capacity life or service life may be less or longer than the bond period.

The paper referred to very properly stresses the fact that rarely less than 50 per cent, and in many systems as much as 80 per cent of the total value of the complete system is involved in transmission and distribution mains, hence the importance of analysing all available data upon service life.

Obsolescence is frequently considered in the depreciation of numerous structures and equipment involved in utilities; but seldom, if ever, has it been applied to the determination of the life of distribution systems. The term "obsolescence" may be applied, by proper qualification, to a definition of the end of useful period of service life, such termination being brought about by the following factors,

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\* Hydraulic and Sanitary Engineer, New York, N. Y.

emphasized in the subject paper, which are not ordinarily considered:

- (1) Changes in location of supply works from one side of the system to the other.
- (2) Increased pressures in many sections of the distribution system.
- (3) Changes in street grades and interference with other underground structures.

In applying such data to other distribution systems there are a number of important considerations, among which are the following:

- (1) The general adequacy of the original system, particularly in reference to weighted diameter of the distribution mains and the hydraulic design of the system.
- (2) Relative quality of water from the standpoint of factors particularly affecting the rate of capacity loss.
- (3) The nature of the expansion of the municipality, for instance, whether primarily representing an increase in population density, or primarily representing an extension of the area served into adjoining territory.
- (4) Relative amount of capital outlay for distribution reinforcement.

From the standpoint of service life in respect to adequate capacity, the policies which have prevailed in each particular system in the past, with regard to minimum size of distribution mains would seem to have a far reaching effect on service life.

The underwriter's report quoted in the paper contains two extremely significant statements, relative to the Springfield system, which are as follows:

- (1) Minor distributors are mainly 8-inch and the considerable amount of 6-inch is mainly located where pressures are high.
- (2) The interior condition of mains is good.

With further reference to the question of size in relation to service life, the charts accompanying the paper although representing a new line of approach, add strikingly to the mass of evidence against the use of 4-inch pipe for water systems. Inasmuch as the average diameter of distribution systems, weighted by length, is generally below 9-inch, minor differentials in size have a major importance to overall service life.

The data presented comprise a strong argument in favor of an adequately high initial expenditure for relatively large mains, for such policy will be reflected in a substantial extension of the service life of such mains.



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## Discussion of the New Law of Design of Cast Iron Pipe

*By W. D. Moore*

**T**HE American Standards Association's Sectional Committee A21 was organized in 1926 and is charged with the responsibility of preparing new standard specifications for cast iron pipe and fittings. The sponsor bodies are American Gas Association, American Society for Testing Materials, American Water Works Association, and New England Water Works Association. Organized under standard A.S.A. procedure, the Committee is composed of representatives, not only from the sponsor bodies and the pipe manufacturers, but also of many other national associations.

After thirteen years of intensive work, Committee A21 has developed a new method of calculating thicknesses of cast iron pipe. This committee has also developed a detailed specification for pit cast pipe, the metal thicknesses of the pipe being calculated according to the new law of design.

Committee A21, at a special meeting in February, 1938, unanimously approved the method of computing pipe thicknesses and its specific application to pit cast pipe then placed before it in detail.

Detailed specifications for cast iron pipe made by processes other than pit cast are now in process of preparation by Committee A21.

In the meantime, the specification for pit cast pipe has been advanced to the position of a proposed standard specification and as such submitted to the sponsor bodies for their consideration.

As a foundation for its work, Committee A21 decided to collect and digest the experience and expert knowledge of the principal users of cast iron pipe throughout the country. A comprehensive questionnaire was sent out and replies received from 97 companies whose

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A paper presented on June 12, 1939 at the Atlantic City Convention by W. D. Moore, President, American Cast Iron Pipe Company, Birmingham, Alabama.

systems embraced more than 30,000 miles of cast iron pipe, ranging in size from 2 to 60 inch. Included in this list were 44 gas companies, 48 water companies and 5 miscellaneous users.

### Data on Breaks of Cast Iron Pipe

Among the most important data developed by the questionnaire were records of breaks of cast iron pipe lines in service. The classification of external cause of failure included excessive earth load, excessive truck load, vibration, bearing on stone ledge or masonry, unequal settlement, and improper foundation with consequent uneven bearing on the soil. A study of this record of cast iron pipe lines in service gave convincing proof that in the great majority of cases where failure occurred, it could not be attributed directly to internal pressure alone. In fact, most of the causes were attributable to external challenges acting alone, or in combination with internal pressure. Failures of pipe 8-inch and smaller were generally cross breaks indicating failure due to beam stress or tensile stress, caused by temperature change, both of which are greatly aggravated by stiff rigid joints. Breaks in the 10-inch and larger pipe were generally split fractures or failures as a ring, indicating that the cause was due to external load acting singly, or in combination with internal pressure. It was evident that pipe lines in the full range of sizes which were not properly tamped to support the pipe were more susceptible to breaks in service than where precautions had been taken to secure good bedding.

It was apparent that the extent of the external load due to the backfill and the influence of the method of laying in affecting the stress concentrations of the pipe were important factors not heretofore generally considered or evaluated in the design of cast iron pipe wall thickness. Formulae previously used for determining wall thickness were all similar, being modifications of the Barlow formula, wherein certain arbitrary additions were made to the wall thickness, calculated solely on the basis of internal pressure.

The Dexter Brackett formula was used by the New England Water Works Association for design of wall thickness of cast iron pipe. The American Water Works Association apparently used the Fanning formula with some minor adjustments. The American Gas Association thicknesses parallel very closely Class "A" thicknesses of the A. W. W. A. All of the above mentioned specifications cover pit cast pipe.

Federal Specification WW-P-421 covers cast iron pipe made by several modern methods of manufacture in which the wall thicknesses are determined by the modified Fairchild formula. In their basic form, all of these formulae are similar and are modifications of the Barlow Formula, as will be noted.

$$\text{Barlow Formula: } t = \frac{PD_1}{2S}$$

$$\text{Brackett Formula: } t = \frac{(P + P_1)r}{3300} + .25$$

$$\text{Fanning Formula: } t = \frac{5(P + P_1)r}{18000} + .333 \left( 1 - \frac{D}{100} \right)$$

$$\text{Modified Fairchild Formula: } t = \frac{5(P + P_1)r}{25000} + \frac{0.28}{D^{0.15}}$$

where for all formulae

$t$  = nominal wall thickness, inches

$P$  = working pressure in lb. per sq. in.

$r$  = radius of bore, inches

$D$  = diameter of bore, inches (nominal pipe size)

$D_1$  = outside diameter, inches

$S$  = tensile strength, lb. per sq. in.

$P_1$  = water hammer allowance, as follows:

	3"-10"	12"	16"	20"	24"	30"	36"	42"-60"
Fanning .....	100	100	100	100	100	100	100	100
Brackett .....	120	110	100	90	85	80	75	70
Fairechild .....	120	110	110	100	95			

In the Brackett formula the divisor is the assumed safe working stress of the metal in lb. per sq. in. In the Fanning and Fairchild formulae, the divisor is the assumed ultimate tensile strength of the iron and a factor of safety of 5 is used.

In all of these formulae, the tensile strength values are assumed strength properties of the pipe. The arbitrary additions were included as an allowance for manufacturing tolerance and corrosion and also as an attempt to provide for earth loads, traffic loads and all other unknowns likely to be encountered in what might be considered as average trench conditions. While the pipe wall thicknesses thus determined have been, in general, satisfactory; in cases of high backfill and unfavorable conditions of laying, they are in-

adequate; whereas in other cases of low and medium heights of cover and well tamped backfill, they are excessive. It is obvious that in the one case pipe designed by such formulae may be unsafe and in the other uneconomical.

### Research Program

Realizing the inadequacy of previous methods of design, Committee A21 undertook to establish a new and rational method of design of cast iron pipe to withstand combinations of external load and internal pressure. It was realized that to do so would entail original research work not only to determine the extent of the backfill loads and truck loads for various heights of cover, but also to establish a rational method of determining the influence of the method of laying to increase or decrease the effective earth load actually transmitted to stress the pipe. In addition, the salient strength properties of the pipe itself must be determined when it is subjected to combined tensile and flexural stresses. A comprehensive series of tests was decided upon to carry out this program and arrangements were made to have the tests conducted at Iowa State College and the University of Illinois.

Iowa State College was excellently equipped to develop the external loads on cast iron pipe lines due to backfill and truck traffic, as they had for many years pursued the study of earth loading on sewer and culvert pipe. Information gained from their previous work was supplemented by tests on several representative sizes of cast iron pipe. Six typical methods of laying cast iron pipe were investigated. These consisted of (1) pipe laid in a flat bottom trench with untamped backfill, designated as "A" condition; (2) pipe laid in a flat bottom trench with tamped backfill, designated as "B" condition; (3) pipe laid on blocks with untamped backfill, designated as "C" condition; (4) pipe laid on blocks with tamped backfill, designated as "D" condition; (5) pipe laid in a shaped bottom trench with untamped backfill, designated as "E" condition, and (6) pipe laid in a shaped bottom trench with tamped backfill, designated as "F" condition. The actual stress to which the pipe was subjected by the earth loads for each method of laying was determined. Ratios were established to convert the earth load to a 3-edge bearing equivalent stress on the pipe for each different method of laying. A significant fact, developed by the tests, is the very appreciable effect that the condition of laying has upon the 3-edge bearing equivalent load. For instance,

the 3-edge bearing equivalent for a 6-inch pipe laid on blocks with untamped backfill is six times as much as when the pipe is laid in a shaped bottom trench with tamped backfill, with all other conditions exactly the same. The difference decreases as the pipe size increases, and for a 60-inch pipe is twice as much. The difference does not follow a straight line relationship, but for all sizes, pipe laid in a shaped bottom trench with tamped backfill is the most favorable trench condition, and pipe laid on blocks with untamped backfill is the worst trench condition.

### **Relationship of Internal Pressure and External Load**

Tests were also conducted at Iowa State College and the University of Illinois to determine the relations of combined tensile and flexural stress to ultimate tensile and bending stresses of cast iron pipe. Several sizes and kinds of pipe, produced as nearly as possible under the same conditions, were tested to determine the ultimate bursting tensile strength of full length pipe when subjected to internal pressure. Pipe were also tested to determine modulus of rupture when subjected only to external crushing load. A sufficient number of tests were made to establish average values. Full length pipes were then tested to destruction with various combinations of internal pressure and external load acting simultaneously. The internal pressure and external load acting simultaneously to cause failure were plotted and it was found that a definite relation existed between them and the ultimate internal and external load when each was acting alone to cause failure. A simple formula was devised to express this relationship.

The salient strength properties of cast iron pipe used to evaluate its ability to resist the forces encountered in service are: first, resistance to internal bursting pressure; and, second, resistance to external crushing loads. A standard test procedure was established by Committee A21 to determine these salient strength properties of the pipe.

The ultimate tensile strength for purposes of design of pipe wall thickness was decided upon as the tensile strength established by bursting tests on full length pipe. This method of determination is significant as contrasted with assumed tensile strength values of the pipe used in previous formulae.

The modulus of rupture for purposes of design of pipe wall thickness was decided upon as the modulus of rupture developed in the

A.S.T.M. Standard 3-edge bearing crushing tests on rings cut from the pipe.

Much work has been done by Committee A21 correlating the mass of tests data. Factors of safety to apply, allowance for water hammer and truck loading as well as allowances for manufacturing tolerances and corrosion have been established by agreement. Methods of test to insure manufacturing control of the physical strength properties of the pipe have been established and rational formulae for the design of pipe wall thicknesses for combination loading based on the ultimate strength properties of the pipe have been evolved. After thirteen years of hard, but very interesting work, a new law of design of cast iron pipe for underground service has been approved by Committee A21.

### Pit Cast Pipe Specifications

The physical strength properties of pit cast pipe as a basis of design have been determined as 11,000 lb. per sq. in. ultimate tensile and 31,000 lb. per sq. in. modulus of rupture. The proposed or tentative specifications for pit cast pipe will soon be published. It will include new classifications of pipe according to wall thickness. The minimum standard thickness class for each size is designated as class 1. Class 2 is 8 per cent thicker than class 1, class 3 is 8 per cent thicker than class 2, and so on, each class having 8 per cent greater wall thickness than the preceding class.

The specifications will include tables designating the class and thickness of pipe required, based on the working pressure, the depth of cover and the type of trench bottom. This will permit quick and easy selection of the safe, and at the same time economical, pipe wall thickness for a wide range of field conditions.

Considerable progress has been made by the manufacturers in the preparation of specifications for pipe produced by the various modern methods of manufacturing pipe, having higher strength properties than pit cast pipe. The pipe referred to are as follows:

- (1) Centrifugal pipe cast in metal molds,
- (2) Centrifugal pipe cast in sand lined molds,
- (3) Pipe cast on the side in stationary green sand molds with green sand cores.

Each of these three cast iron products has its own combination of physical strength properties, i.e., ultimate tensile strength in full length bursting, and modulus of rupture of a ring cut from the pipe.

However, for design purposes to determine a metal thickness which can be applied to any one or all of these products, it is necessary to use the lowest tensile strength factor of the three products and also the lowest modulus of rupture. Obviously, when pipe are furnished with higher physical strength values and the same metal thickness is used, this pipe will have a greater factor of safety against stress or corrosion. The minimum physical strength properties applicable as common factors to the whole group of pipe produced by the several modern methods of manufacture have been tentatively agreed upon as 18,000 lb. per sq. in. ultimate tensile and 40,000 lb. per sq. in. modulus of rupture. The methods of determining the actual strength properties of the pipe are fully set forth in the new law and when pipe having higher strength properties are made available, full advantage can be taken of such superior strength since the same law of design of wall thickness will apply. The new law of design is an entirely different approach to the calculation of cast iron pipe wall thickness. Many of the principles are based on original research work. Of necessity, it embraces technical data not contained in text books nor familiar to more than a very few people, confined until recently largely to members of Committee A21. A very excellent paper was presented at the 1938 A.W.W.A. Convention in New Orleans on the subject "A Proposed New Method for Determining Barrel Thickness of Cast Iron Pipe" by Thomas H. Wiggin, M. L. Enger, and W. J. Schlick and was published in the May, 1939, issue of *Journal of the American Water Works Association*. It very thoroughly deals with the broad considerations of Committee A21 in evolving the new law of design and gives the detailed steps necessary in determining pipe wall thickness. The author presented a paper before the Distribution Conference of the American Gas Association at Chicago, April, 1939, which included as an addendum a paper entitled "Explanation of the New Law of Design for Cast Iron Pipe as Developed by Committee A21 of American Standards Association," prepared by Research Department, American Cast Iron Pipe Company. This paper contains tables and graphs enabling quick and easy determination of pipe wall thickness for field conditions likely to be encountered in service and gives all of the basic information necessary for thickness determinations.\*

It is beyond the scope of this paper to explain in detail the procedure and technical steps necessary in using this new law of design.

\*A copy of this treatise will be furnished by American Cast Iron Pipe Company to interested persons on request.

The separate publications on the law itself referred to will give such information to those who are interested. The purpose of this paper is to discuss some of the highlights of the law and to show for some typical cases the wall thicknesses determined by the new law compared with the well-known A.W.W.A. standard and Federal specification standard thicknesses.

The new law of design emphasizes the importance of external loading of the pipe line and the fact that the external load conditions as well as internal pressure must be considered if safe, and at the same time economical design of wall thickness is to be obtained.

### Factors of Safety

Allowances for truck traffic are made based on the load of two 5-ton passing trucks. For water pipe, pressure additions are also made for water hammer whenever the wall thickness required for such water hammer effect exceeds the wall thickness required to resist the truck load. Allowances for both are not made as the chance that they would occur simultaneously is extremely remote. In general, for heights of cover of less than 5 ft., the truck load is the dominating influence and for heights of cover 5 ft. and more, the water hammer effect dominates. Obviously, pipe for gas service does not take account of water hammer effect and the truck load is included for all heights of cover. The truck load on the pipe decreases rapidly as the height of cover increases.

The factor of safety agreed upon is  $2\frac{1}{2}$  for both external loads and internal pressure acting simultaneously, which also includes the allowance for truck loading or water hammer, as the case may require. These factors of safety apply in calculating the net wall thickness required to resist all of the forces acting simultaneously on the pipe line. To the net thickness thus determined, additions are made as an allowance for manufacturing tolerance and an allowance for corrosion. The factor of safety of  $2\frac{1}{2}$  applies to the external load and internal pressure acting simultaneously, using the net thickness, and should not be confused with factors of safety usually thought of as applicable to the thin tube formula, considered only from the standpoint of internal pressure and ultimate tensile strength. This may be clarified by reference to figure 1 which shows for a given pipe diameter and net wall thickness the ultimate bursting pressure,  $P$ , that will cause failure of the pipe when the external load is zero and the ultimate external load,  $W$ , that will cause failure of the pipe when

the internal pressure is zero. These ultimate values are expressed as 100 per cent. The parabolic curve indicates the percentage of these ultimate forces that will cause failure when the pipe is subjected to internal pressure and external loads acting simultaneously. The forces which, acting simultaneously, will cause failure, are indicated as  $w$  and  $p$ .

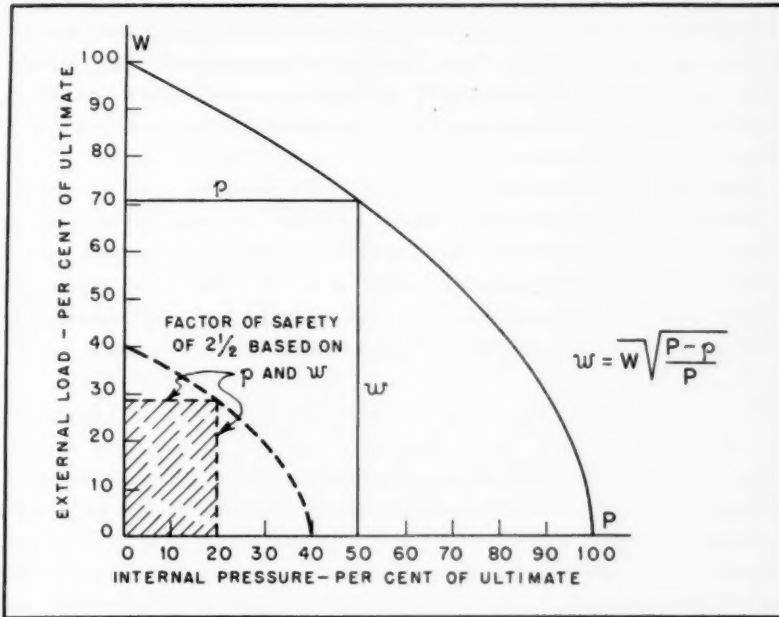


FIG. 1. Application of the External Load, Internal Pressure and Safety Factor to the Design of Cast Iron Pipe.

NOTE:  $W$  = external load required to crack pipe when internal pressure is zero.

$P$  = internal pressure required to burst pipe when there is no external load.

$w$  = external load which will cause failure when internal pressure is  $p$  for a given pipe diameter and net wall thickness.

The large solid line rectangle indicates a particular combination of internal pressure and external load acting simultaneously that will just cause failure of the pipe. The small dashed line shaded rectangle indicates the percentage of ultimate strength values to which the pipe wall metal is subjected when a factor of safety of  $2\frac{1}{2}$  is used based on this combination of forces acting simultaneously. It will be ob-

served that the pipe wall metal is stressed to only 20 per cent of its ultimate bursting strength and 29 per cent of its ultimate external load strength. The indicated factor of safety for this particular case is, therefore, 5, if based on ultimate bursting pressure acting alone. Furthermore, this factor of safety is based on the net thickness before adding any allowances for manufacturing tolerances and corrosion. Obviously, the indicated factor of safety would be greater if based on the nominal thickness. Modern pipe designed by the new law for the typical condition of 100 pounds working pressure, 5-foot cover with tamped backfill has theoretical factors of safety based on nominal thickness and working pressure alone of 12 or more for sizes 12-inch and under, and 7 or more for the larger sizes.

Arbitrary minimum standard thicknesses have been adopted on the basis of judgment from the standpoint of hazards in transportation and feasibility of manufacture. In the smaller pipe sizes for mild pressure and trench load conditions, the calculated thickness in some cases may be appreciably less than the minimum standard thickness. In such cases, the minimum standard class thickness will govern.

Pipe thicknesses designed by the new law have been investigated from the standpoint of beam loading of individual lengths. In all cases, except 3-, 4- and 6-inch pipe, when laid deep on blocks with untamped backfill, the wall thicknesses calculated by the new law are sufficient. For these few exceptional cases, proper notation will be made in the tables of rating. The stresses provided for in the new law of design embrace only those stresses that may be imparted to individual lengths of pipe considered as a unit. Obviously, it is not economically practical to provide wall thickness to resist expansion and contraction stresses that may be cumulative to cause trouble in small diameter pipe lines through the use of stiff, rigid joints. Nor is it practical to provide wall thickness to resist excessive beam loads acting over long spans that may result from rigid joints, if the pipe line is caused to shift in service due to earth subsidences or uneven settlement. The logical way to provide for the effect of temperature changes and probable shifting of pipe lines in service is to prevent as far as possible the imposition of superimposed stresses to the pipe. This emphasizes the desirability of good bedding and proper tamping of the pipe line and the use of joints of adequate strength that provide for longitudinal movement and lateral deflection between adjacent pipe lengths, such as the bell and spigot joint and flexible mechanical joints.

Among the outstanding facts revealed by the tests and incorporated in the new law of design are the effects of the height of cover and method of laying upon the required pipe wall thickness. As pointed out previously, pipe laid in a shaped bottom trench with tamped backfill is the most favorable trench condition and pipe laid on blocks with untamped backfill is the worst trench condition. Regardless of the type of trench bottom the economy or increased factor of safety obtained by tamping the backfill is so pronounced that apparently conditions will be rare, particularly for water distribution systems, which will justify any other procedure. It may be well to point out here, that the benefits of the tamped backfill are obtained when it is tamped under and around the pipe up to a few inches above the pipe. While the shaped bottom trench gives the most favorable trench condition, it is not generally economical or practical except in special cases of large diameter pipe. Laying the pipe in a flat bottom trench with a tamped backfill gives the second best trench condition. It is a thoroughly practical and economical method of installation and may be considered as the standard trench condition for water distribution mains.

### Graphs Developed

The accompanying graphs illustrate the effect of the cover and method of laying, as well as the internal pressure, on the required pipe wall thickness. Thicknesses calculated by the new law of design are compared with A. W. W. A. Class B thicknesses and Federal class 150 thicknesses calculated by the modified Fairchild formula.

As shown in the graphs, the thicknesses for pipe by the new law of design are as calculated for sizes 14-inch and larger and as adjusted to the nearest standard class thickness for sizes 12-inch and smaller. The difference between the calculated thickness and the nearest standard class thickness is negligible in the larger sizes, although it may be appreciable in the smaller sizes due to the arbitrary minimum thicknesses which have been established as previously mentioned.

Figure 2 shows thicknesses for pit cast pipe required by the new law of design for 100 lb. water pressure compared with A. W. W. A. Standard Class B pipe having a pressure rating of 86 lb. Pipe designed for 100 lb. water pressure, when laid in a flat bottom trench with tamped backfill (trench condition B) and 5 ft. of cover may be considered as suitable for the great majority of water distribution main installations. It will be observed that wall thicknesses of pipe designed by the new law for this typical condition are slightly

less than A. W. W. A. Class B thicknesses. For the unusual condition of pipe laid on blocks with untamped backfill the required thicknesses by the new law are greater than Class B and when laid on blocks with tamped backfill they are substantially the same as Class B. This graph also indicates the effect of the method of laying on the required pipe wall thickness when all other factors are the same. It emphasizes the benefits of good bedding for the pipe and proper tamping of the backfill.

Figure 3 shows pipe wall thicknesses for pit cast pipe designed by the new law for the typical condition of 100 lb. water pressure, trench

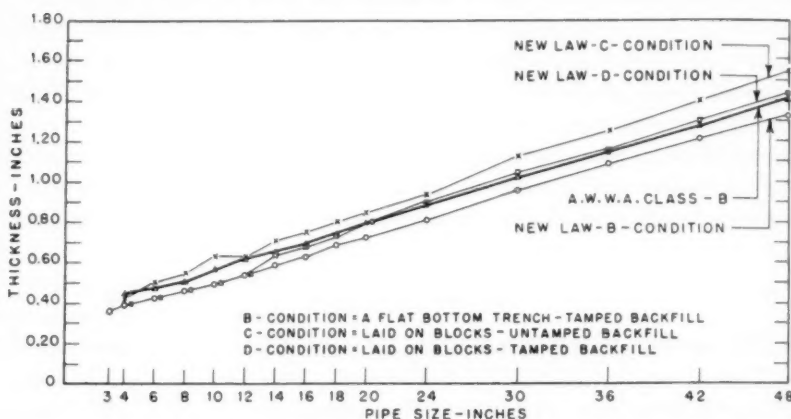


FIG. 2. Comparison of Pipe Wall Thicknesses; A. W. W. A. Standard Class B vs. New Law 100-pound Water Pressure, 5-foot Cover, for pit cast pipe of 11,000 lb. per sq. in. tensile and 31,000 lb. per sq. in. modulus rupture.

condition B, with 5 ft. of cover and also the unusual condition of 16 ft. of cover compared with A. W. W. A. Class B thicknesses. This graph illustrates the effect of the depth of trench on the required pipe wall thickness.

Figure 4 shows wall thicknesses for pipe produced by modern methods of manufacture having higher strength properties than pit cast pipe. Thicknesses designed by the new law for the typical condition of 100 lb. water pressure, trench condition B with 5 ft. of cover and also the unusual condition of 16 ft. of cover are compared with Federal Class 150 thicknesses. The effect of cover is emphasized indicating that, in general, Federal Class 150 thicknesses are thicker than necessary for standard conditions of laying but are

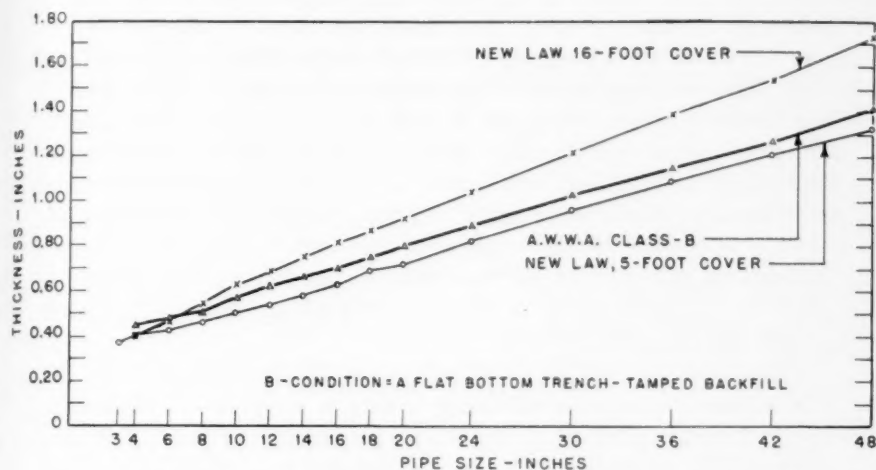


FIG. 3. Comparison of Pipe Wall Thicknesses; A. W. W. A. Standard Class B vs. New Law 100-pound Water Pressure, B Condition, for pit cast pipe of 11,000 lb. per sq. in. tensile and 31,000 lb. per sq. in. modulus rupture.

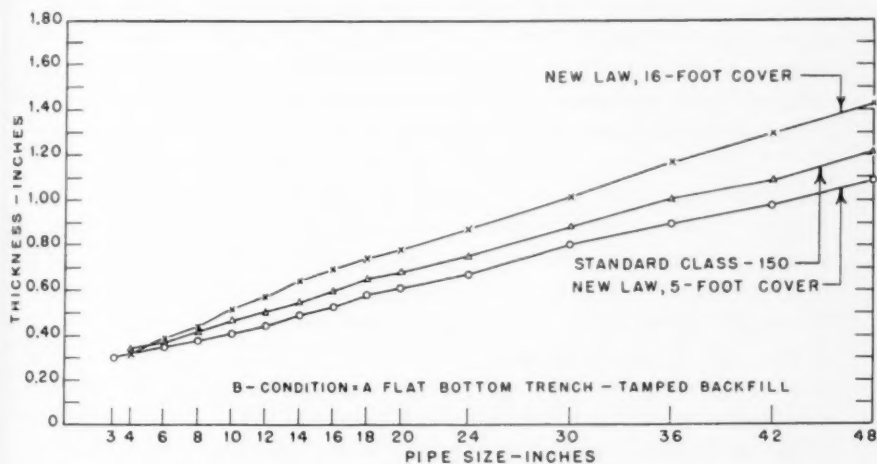


FIG. 4. Comparison of Pipe Wall Thicknesses; Standard Class 150 vs. New Law 100-pound Water Pressure, B Condition, for modern pipe of 18,000 lb. per sq. in. tensile and 40,000 lb. per sq. in. modulus rupture.

inadequate for the very unusual condition of 16 ft. of cover with 100 lb. working pressure.

Figure 5 shows pipe wall thicknesses for pit cast pipe and pipe

produced by modern methods designed by the new law for 50 lb. and 150 lb. working pressure, trench condition B with 5 ft. of cover. Also thicknesses are shown for pit cast pipe designed for 50 lb. working pressure, trench condition B with 16 ft. of cover. This graph indicates the effect of pressure when all other conditions are the same and shows the relatively small effect of pressure on wall thickness for pipe having the higher physical strength properties. The effect of the height of cover is again emphasized as it will be observed that the increase in thickness of pit cast pipe required when the height of cover is increased from 5 to 16 ft. is greater than when the pressure is increased from 50 to 150 lb.

Figure 6 shows pipe wall thicknesses for the typical case of 100 lb. water pressure, trench condition B with 5 ft. of cover. Thicknesses by the new law of design are shown for pipe metals of different physical strength properties. The line designated as "11,000 lb. per sq. in. Tensile—31,000 lb. per sq. in. Modulus of Rupture" is applicable to pipe produced by the pit cast method. The line designated as "18,000 lb. per sq. in. Tensile—40,000 lb. per sq. in. Modulus of Rupture" represents the strength properties tentatively agreed upon as common factors for purpose of design of pipe produced by the several modern methods of manufacture considered as a group. As previously explained, they represent the lowest combination of strength properties of the whole group of pipe. The line designated as "25,000 lb. per sq. in. Tensile—50,000 lb. per sq. in. Modulus of Rupture" represents a combination of higher strength properties that is being produced commercially at the present time. This graph indicates the economies in thickness that may be obtained with the higher strength metal or the greater factor of safety against stress and corrosion that is inherent in such superior metal when pipe of the same thickness is used.

A study of these few graphs shows the marked effect that field conditions other than the operating pressure have upon the wall thickness required in cast iron pipe. They indicate that pipe designed by previous formulae on the basis of pressure alone may for some trench conditions be uneconomical and for other trench conditions inadequate. At the same time, the graphs indicate that for usual or average field conditions, pipe thicknesses required by the new law of design are not a radical departure from existing standards.

The new law of design of cast iron pipe is the culmination of years of study by a collection of some of the best minds in America,

THICKNESS - INCHES

THICKNESS - INCHES

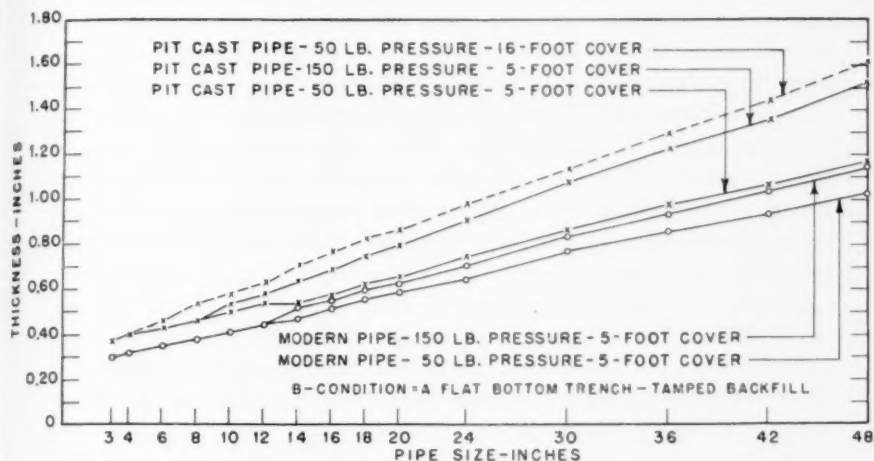


FIG. 5. Comparison of Pipe Wall Thicknesses; New Law 50-pound Water Pressure, B Condition, vs. 150-pound Water Pressure, 5-foot Cover vs. 16-foot Cover, for pit cast pipe of 11,000 lb. per sq. in. tensile and 31,000 lb. per sq. in. modulus rupture, and for modern pipe of 18,000 lb. per sq. in. tensile and 40,000 lb. per sq. in. modulus rupture.

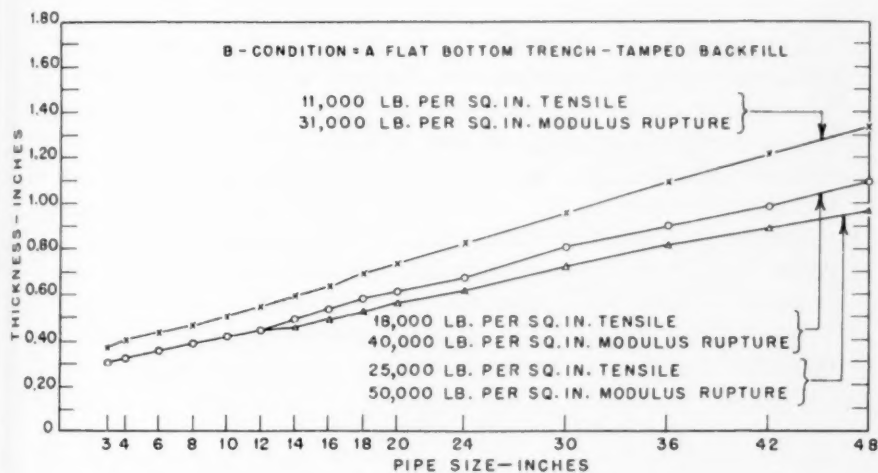


FIG. 6. Comparison of Pipe Wall Thicknesses; New Law 100-pound Water Pressure, 5-foot Cover, B Condition, Iron of Different Strength Properties.

based on a most thorough and comprehensive research test program. It establishes the method by which the strength properties of cast iron pipe to resist the combination of forces encountered in service may be determined. It establishes a rational method of calculating to a high degree of accuracy the extent of the external loads encountered in service, and their effect to stress the pipe as influenced by the method of laying. Cast iron pipe may now be selected for known or assumed field conditions with assurance that the selection is both safe and economically sound.

The law of design as developed by Committee A21 is directly applicable to the design of cast iron pipe in underground service. It sets a precedent, however, which should be followed in establishing the strength properties and behavior of any other materials which may be considered for service in this field.

The basic fundamentals of the law are applicable to the design of cast iron pipe by whatever method of manufacture it may be produced. Full advantage may be taken of the high physical strength values of pipe produced by present modern methods of manufacture or superior strength properties that may be developed in the future. Thus, the highest possible economy in the selection of cast iron pipe is assured.

**Discussion by D. B. Stokes.\*** In Messrs. Wiggin, Enger and Schlick's paper, "A Proposed New Method for Determining Barrel Thickness of Cast Iron Pipe" (Jour. A. W. W. A., **31**: 841 (1939)), the nomogram shown in fig. 34, page 900, was used to obtain the net computed thicknesses. Such a nomogram can only be used to obtain the pipe thickness for the one combination of bursting tensile and ring crushing strengths for which it is drawn. An alternate method which can be used for any combination of bursting tensile and ring crushing strengths will be presented.†

When  $2tS/d$  and  $Rt^2/.0795(d + t)$  are substituted for  $P$  and  $W$  respectively in the pipe thickness formula

$$P - p = Pw^2/W^2$$

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\* Vice-President, United States Pipe and Foundry Co., Burlington, N. J.

† This method was developed by R. B. Hutchinson, Research Dept., U. S. Pipe and Foundry Co., Burlington, N. J.

the following expression can be obtained for  $t$ :

$$t = pd/2S + \left( \frac{.0795wd}{R} \right)^2 \left( 1 + \frac{t}{d} \right)^2 (1/t^3)$$

For any values of  $t$  and  $(.0795wd/R) (1 + \frac{t}{d})$  there will be a definite  $pd/2S$ . Therefore, if these three variables are combined in a graph a solution for the thickness can be obtained therefrom. Two such graphs, one for thicknesses up to .90 in. and the other for thicknesses above .90 in. are shown. On these graphs the abscissae represent values of  $t$ , the ordinates values of  $(.0795wd/R) (1 + t/d)$  and the curved lines values of  $pd/2S$ .

Using the same example as that given in Messrs. Wiggin, Enger and Schlick's paper, that is a 20-inch pit cast pipe; a 100-pound static pressure; a 90-pound water hammer; 5-foot earth cover; laying condition B, the method of obtaining the pipe thickness by means of these graphs will be explained.

Case 1. Water hammer used but no truck load.

Design pressure =  $2.5 (100 + 90) = 475$  lb./sq.in.

Design load =  $2.5 \times 1,760/1.47 = 2,990$  lb./lin.ft.

$pd/2S = 475 \times 20/22,000 = .432$

For an approximation the  $t$  in the term  $1 + t/d$  can be assumed to be the next even tenth of an inch above the value of  $pd/2S$ .

$$(.0795wd/R) (1 + t/d) (t = .5) = (.0795 \times 2,990 \times 20/31000) \times (1.025) = .1570$$

On the graph follow the horizontal line for  $(.0795wd/R) (1 + t/d) = .1570$  until it intersects the curve for  $pd/2S = .432$ , this gives a thickness of .57.

Using  $t = .57$  instead of the assumed .50 makes  $(.0795wd/R) (1 + t/d) = .1578$  instead of .1570. This does not increase the required net thickness an appreciable amount as it is still .57 in.

Case 2. Truck load but no water hammer.

Internal pressure = 250 lb./sq.in.

Crushing load = 4,030 lb./lin.ft.

$pd/2S = 250 \times 20/22,000 = .227$

$(.0795wd/R) (1 + t/d) (t \text{ assumed} = .3) = .2065$

First approximation net thickness = .52 in.

$(.0795wd/R) (1 + t/d) (t = .52) = .2085$

Net thickness for  $pd/2S = .227$  and

$(.0795wd/R) (1 + t/d) = .2085$  is .53 in.

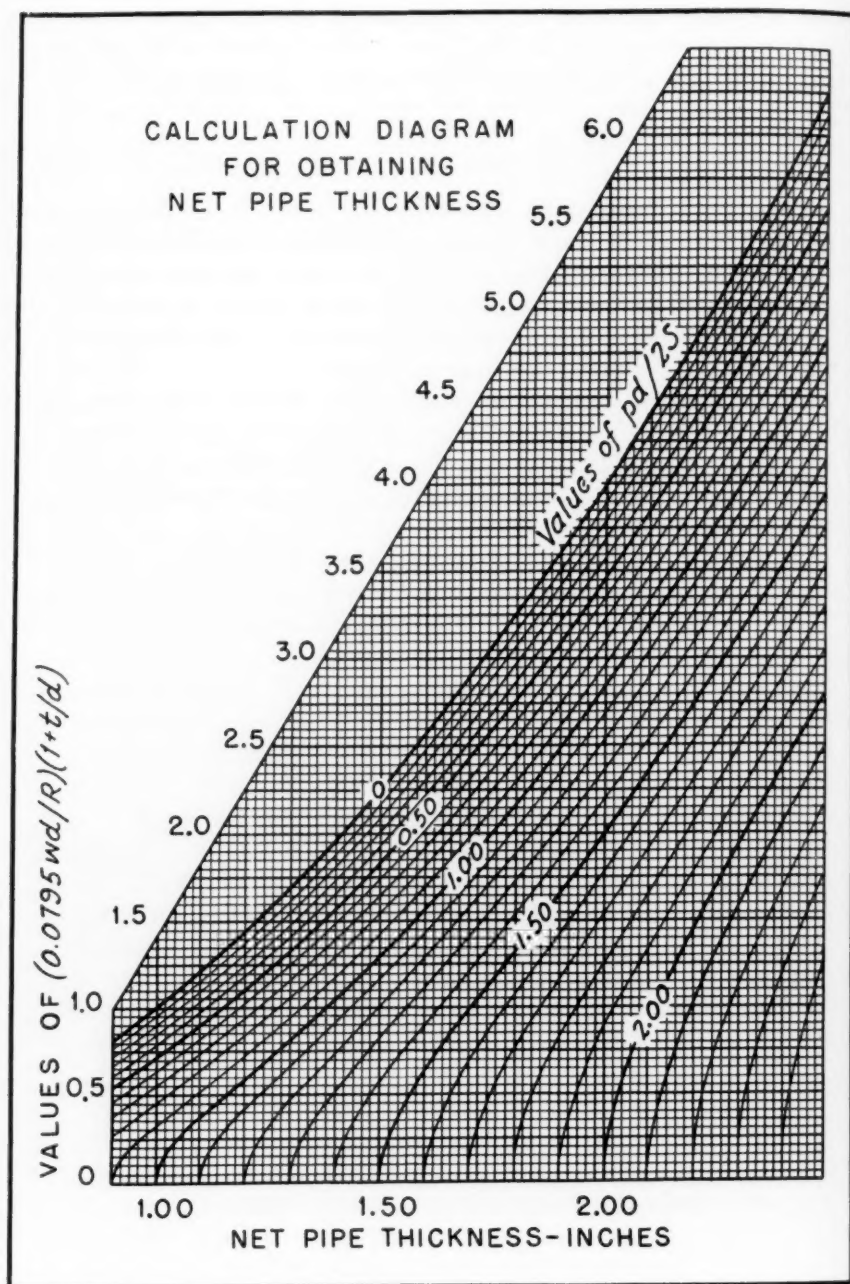


FIG. 1

These net thicknesses for Cases 1 and 2 are the same as those obtained from the nomogram.

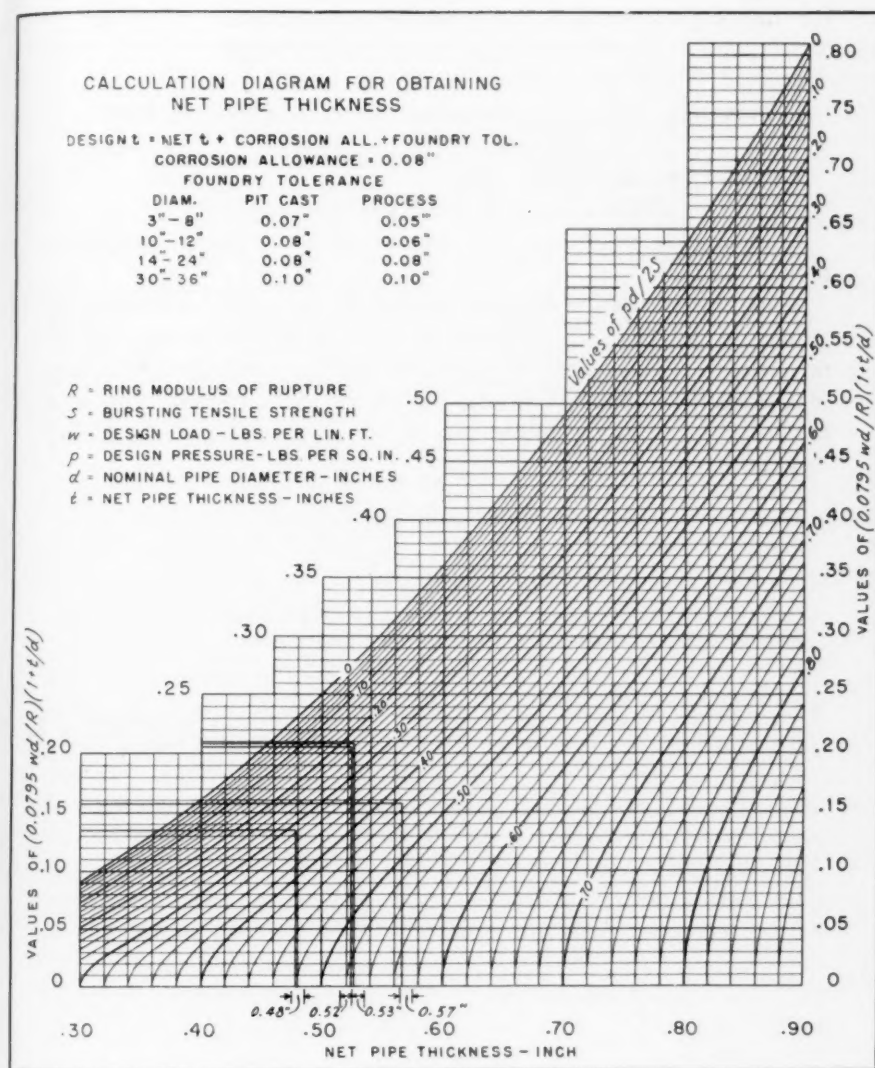


FIG. 2

If the pipe strengths were changed to 15,000 lb. per sq.in. bursting tensile strength and 36,000 lb. per sq.in. ring modulus of rupture

this calculation diagram could still be used while the nomogram would be useless.

Using Case 1 as an example of calculating the required thickness for these higher strengths the procedure is as follows:

$$p = 475 \text{ lb./sq.in.}, w = 2990 \text{ lb./lin.ft.}$$

$$pd/2S = 475 \times 20/30,000 = .317$$

$$(.0795wd/R) (1 + t/d) \text{ (Assume } t = .4) = (.0795 \times 2990 \times 20/36,000) (1.02) = .1347$$

Net  $t$  by first assumption .48 in.

$$(.0795wd/R) (1 + t/d) (t = .48) = .1352$$

Changing the assumed  $t$  to the actual  $t$  does not change the thickness of .48 in.

Using the calculation diagram requires the extra step of calculating values of  $pd/2S$  and  $(.0795wd/R) (1 + t/d)$  but it has several advantages. These are facilitation of interpolation, demonstration of the relative effect of internal pressure and external load, and applicability to any combination of bursting tensile strength and ring modulus of rupture.



## Centrifugal Pumps Designed for Underground Waters

*By A. O. Fabrin*

A DEEP well turbine pump may be used successfully to elevate ground water only when there is an understanding both of centrifugal pumps and of the peculiar characteristics of the well in which the pump must operate. It is not sufficient to consider only the ability to control centrifugal force in well pumping problems; it is also necessary to consider every project as a special problem with respect to well performance. Wells are not just holes in the ground, or at least should not be so considered or treated.

The pump manufacturer has the advantage over the contractor who "manufacturers" the well. The pump is a machine that reacts to certain laws and can be constructed and tested in a laboratory where accurate instruments measure its quality. The well, however, is not a machine and cannot be produced in a factory, but it entails just as much care if not more than in the case of the pump, because the contractor cannot have at his command materials of well-defined composition. Instead he must rely upon variable geological formations to produce the water supply.

Each successive year sees increasing interest in well construction. Statutes providing specifications for sanitary types of wells have been enacted in many states and cities in this country. Aside from sanitary precautions which are so necessary, the statutes frequently provide for controlling the amount of water taken from the natural underground reservoir. The popularity of well water for air-conditioning has increased the load on city sewer systems because of having to dispose of spent conditioning water. All of these factors have forced to the front the necessity of giving well constructors the consideration which they have so long merited.

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A paper presented on April 11, 1939 at the Kentucky-Tennessee Section Meeting at Memphis, Tennessee, by A. O. Fabrin, Chief Engineer, Layne & Bowler, Inc., Memphis.

Improved well construction is welcome to the centrifugal pump engineer, because the well is truly the extended suction portion of the centrifugal pump. Since about 75 per cent of pump troubles are found on the suction side, it follows that the ideal well for the centrifugal pump is one wherein the water level, when pumping, remains quiet and free from turbulence.

Specifications that pertain only to the centrifugal pump which is to be installed in a well oftentimes are taken up solely with the detail of the pump dimensions such as shaft size, type of bearings, span between bearings, size of column, etc. These proportions should be left to the pump manufacturer since his experience, if it extends over a long period of time, makes him better qualified to draw from his practice the correct solution.

The manufacturer's guarantee on his own standard construction is one without mental reservation. If he must guarantee construction details not exactly to his standard, he feels less certain of the results, but of course the customer never will know that.

It is much more important to supply the bidder with all data on the well characteristics. While the centrifugal pump may appear highly complex in its workings it is not as erratic in performance as many would have us believe. The relations between vane angles and peripheral velocity of the impeller and entrance areas may seem intricate but those concerned with their determination base their selection on technical knowledge and practical experience. Naturally, the broader this experience when coupled with a knowledge of present day theory the more satisfactory is the final design. Of course this accumulation of data is considered one of the assets of the pump manufacturer. Since experience is always in the process of evolution, it follows that the early turbine pumps often fell short of predicted performance. Some of us still have memories of the days when the fantastic claims of performance required brutal application of large allowances in order to reconcile practice with promise.

Turbine pumps for well work, while they are of the centrifugal type, must be treated differently than the horizontal centrifugal pump designed for operation above ground. If one compares the two types of pumps, that is, the turbine centrifugal for wells with the horizontal pump, it becomes obvious that the well pump, because of space limitations utilizes higher velocities than the horizontal pump when delivering a given quantity of water against a given head. High velocity stream flow in curved closed pump passages requires

careful design, because the efficient recovery of pressure is dependent upon the correct rates of positive and negative acceleration of the water stream. The impeller being the only moving part of the pump positively accelerates the stream flow. The stationary vanes in the pump casing form channels which, because of their proportions, negatively accelerate the water so that the velocity head generated by the impeller is partially converted into pressure head.

The impeller, since it is the working part of the pump, must have entrance and exit angles correctly associated so that the cross section of the water passages will vary properly in order to realize the desirable rate of positive acceleration through the impeller. Noisy operation and vibration, the result of cavitation, will result unless the best proportions are present in the impeller. Often these noticeable disturbances are companions of corrosion or erosion. Even in the event ideal proportions for the pump have been attained, the performance proven in the testing laboratory may never be realized if the suction conditions in the field are at fault.

Recent tests and investigations indicate that the fault is not all that of the pump whenever field performance varies greatly from factory guarantees.

### Pump Testing Routine

It should not be difficult to convince a skeptic that a manufacturer of pumps cannot afford to publish test data of questionable value. After all, a pump test is not complicated when conducted in a laboratory equipped with standard instruments. Test procedure is standardized and if the question is raised as to test results it reverts to instrument accuracy and care in reading. The questions can be satisfied by calibrating the instruments and checking or verifying the readings by several observers.

In the laboratory the pump is subjected to different pressures by gate valve regulation at the discharge outlet. For each valve setting a different capacity is delivered. From these data a head capacity curve is constructed.

When the pump is installed in a well where the distance from the active level of water to the surface is known, we can, by referring to the head capacity curve, determine the output of the unit. For example, consider a well and a pump operating with an active water level 200 feet below ground. If the water is discharged into a reservoir at the surface, the total head on the pump, neglecting

column friction, will be 200 feet. By referring to the head capacity curve of the pump we can determine what the output of the pump and well should be.

Figure 1 illustrates the foregoing statement. In reality fig. 1 consists of the well characteristic super-imposed upon the head capacity characteristic of the pump. This chart is based on a well with static water level of 50 feet, that is to say, when no water is being pumped, the water rests 50 feet below ground. If a test pump equipped with a valve at the surface outlet has been installed in the well, readings

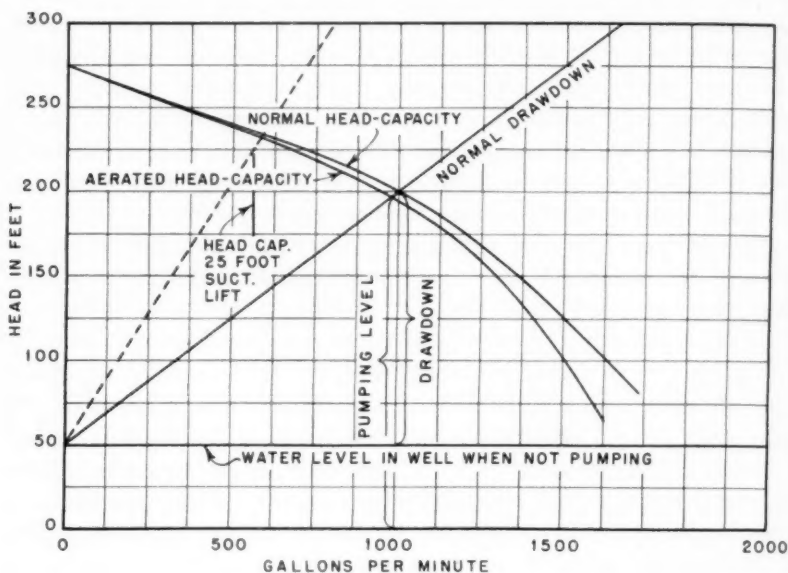


FIG. 1

of pumping level can be obtained for various capacities. In general the drawdown is more or less in direct proportion to the capacity, for instance at 500 gallons per minute the water level receded 75 feet deeper into the ground while at 1,000 gallons per minute the recession stopped at 150 feet below the static level. Presenting this in graphical form the results are a straight line, labeled drawdown curve on fig. 1. Super-imposed on this chart is the head capacity curve of the pump. At the capacity where the head curve of the pump crosses the drawdown curve of the well we reach a state of balance. If the total head measurements match the capacity measurements derived

from the laboratory test, then we can be sure the well is constructed properly, insuring a copious supply of non-aerated water.

Aerated water affects pump performance as you will see by referring to fig. 2.

The capacity for given heads has been decreased and the point of interception on the drawdown curve has moved over towards the left. To say the pump has failed to meet the guarantee is a mistake or misstatement. The correct statement would be that the well and pump are not properly associated. As far as dimensions are con-

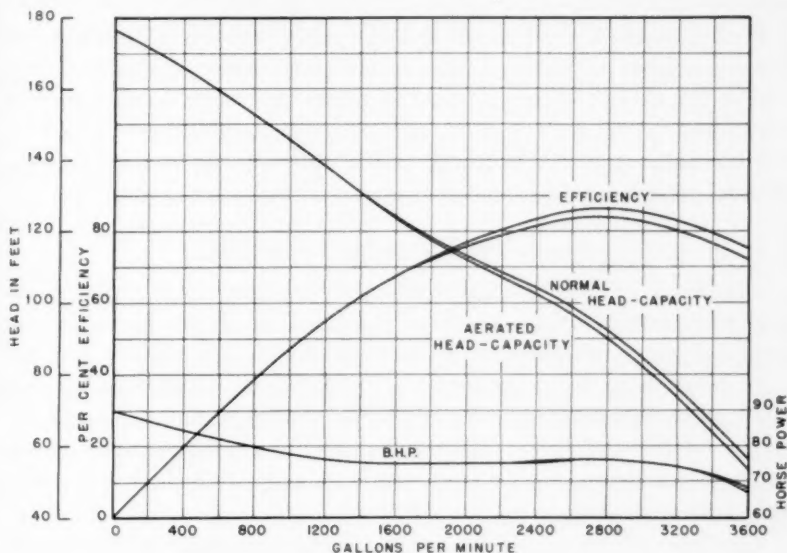


FIG. 2. Effect of air in water on performance of deep well turbine pump; amount of air in water unknown.

cerned, the well casing may be correct, the water bearing sands may be ideal for a given pump demand, but the type of screen used and the manner of placing the screen have a lot to do with the combined performance of well and pump.

Since the trouble analysis of an installed turbine well pump oftentimes is speculative, it also becomes fertile ground for debate. Recently the opportunity to observe the effect of aerated water was made possible through a test conducted in the laboratory. The test was not purposely conducted on aerated water, but accidentally the water in the suction well in the laboratory had been lowered and when

plotting the results from readings it was noticed that the head capacity curve did not match that of several tests conducted on identically the same size and type pumps. The pump was removed from the laboratory and inspected for faulty assembly or for possible obstructions in the suction pipe or impellers. Nothing was found to cause any possible trouble and the pump was again placed in the laboratory for retest. The water level in the suction well had been raised in the meantime and the retest now developed readings rational and consistently in accordance with the standard tests. The tests on both conditions of operation are shown in fig. 2.

Both tests were conducted with the same pump setting and with the water level in the suction well above the upper impeller. The end of the suction pipe was six feet below the water on the aerated water test. The water on the aerated run cascaded three feet into the ten-foot-square suction well and in spite of the six-foot suction pipe submergence, entrained air found its way into the pump.

#### **Conditions Producing Excess Air**

The deep well centrifugal pump will continue to function when handling aerated water provided the impeller of the first stage or lowest stage is submerged. In fact most deep well pump manufacturers state that their guarantees are based on the conditions wherein the lowest stage is completely submerged and that the water supplied to the suction of the pump is free from gases or is non-aerated. These precautions are necessary when the well and pump are awarded to two different contractors.

Of numerous well conditions that make for aeration, one is created by taking a water supply through well screens from two sand strata located at different levels and blanking the intermediate non-producing strata with well casing. Such a condition is conducive to water aeration when the active water level is below the upper screen so that the supply from the top screen must fall through space impinging upon the protruding pump shoulders while it is descending to the active level. In the descent the water being broken up absorbs air. As mentioned before, the deep well pump will function when some air is entrained in the water but the life of a pump operating thus is shortened. The corrosion rate is accelerated when free oxygen is carried in the water stream. Further if the water contains iron and carbon dioxide, the discharge column above the static water level will be coated with layers of insoluble ferric hydroxide. In

time the cross-section area of the conduit will be decreased to such an extent that the friction loss runs up the cost of pumping and the yield in capacity will suffer. In the pump bowl the velocity of flow is great enough to prevent deposits of ferric hydroxide but the oxygen, because of the absence of this insulating medium, will combine with the hydrogen and accelerate the corrosion rate. If by reason of design there are pockets or spaces for dead water to rest, or if eddies are present in these places, ferric hydroxide may be deposited and eventually clog or build up sufficiently to contact the moving impeller and introduce excessive power demands. There is also the condition of well pumping where the active water level in the well screen is low enough to permit some of the screen openings to discharge water into the well above the pumping level.

#### **Continued Aeration Affects Yield**

Continued aeration will not only affect the pump but will also tend to cause the screen openings to become clogged with iron deposits. As this action progresses the specific yield of the well decreases and the pumping level of course lowers. Since the head and capacity of a centrifugal pump are intimately related and dependent upon each other, it follows that as the total head increases the capacity falls off.

When the capacity of the well, for a given pumping level, decreases because of deposits in the screen openings, the drawdown curve of the well will change and the normal curve no longer is correct. Now we have a curve shown by dotted lines on fig. 1 which illustrates that the pumping levels have been increased.

For example, where the well drawdown at 525 gallons per minute was 85 ft. when the well was not affected, it has now increased to 175 feet.

The distance from the surface to the active water level is now 225 feet when 525 gallons per minute is being delivered. If the pump setting is 200 feet from the surface down to the lowest impeller, it is obvious that the pump will be operating with a 25-foot suction lift.

The head increase has been affected on the suction side of the unit and we naturally are interested in the characteristics of a deep well pump when operating under a vacuum. Deep well pumps are not generally considered capable of operating with a vacuum condition existing on the suction side.

A series of tests has been conducted with arrangements to produce lifts under three different vacuum conditions. All tests were conducted with 20 feet of suction pipe and circular orifices attached to the bottom of the pipes. The vacuum connection was located about three feet below the lowest impeller. From here a pipe was directed up to the U-tube at the pump base.

When the pump was started the water level was above the lowest impeller to eliminate priming. Then, as the unit was brought up to speed, the water level in the suction pit was lowered to one foot

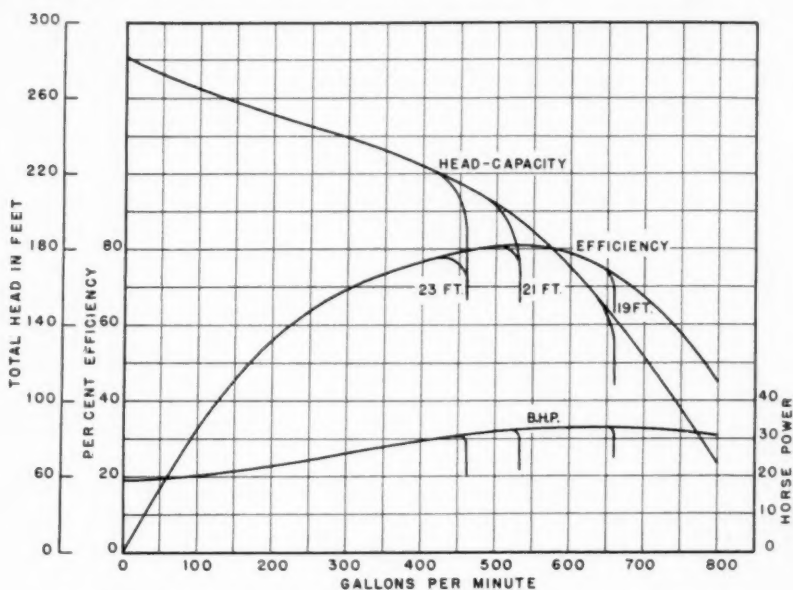


FIG. 3. Tests on deep well turbine pump when subjected to various negative suction lifts.

below the vacuum connection. These tests were not conducted to set a precedent for deep well pump application under suction lift conditions. They were conducted to demonstrate that the turbine pump will, under proper conditions, "pull a vacuum," and to correct the erroneous belief that the turbine well pump cannot operate when the pumping level or active level is below the lowest stage.

If a deep well pump suffers in capacity or efficiency when the water is only a short distance below the lowest impeller, it is because air or gas has entered into the suction pipe. This is due to air entrained

in water cascading to the active level or a short suction pipe and whirlpools causing air to interfere with a solid supply of water.

The tests on suction lifts, as shown in fig. 3, clearly prove that if the water entering the pump is taken from a quiet source and non-aerated, excellent performance can be realized even though the unit operates under a vacuum. In reality the deep well centrifugal is as fool-proof as the horizontal centrifugal and will, under proper conditions work under almost the same suction lift. I say "almost" because the space limitations imposed upon the design necessitate higher water velocities at the center of the impeller where all the water to be pumped must enter. Every foot of velocity at the center of the impeller detracts from the "lifting power" of the pump. By inspecting fig. 3, it becomes obvious that refinement in design has insured excellent performance in spite of the space handicap which is not so restricting in horizontal pumps.

Investigations conducted by others show that no noise is caused by a small amount of air entrained in the suction, but when over 2 per cent of air by volume is encountered the efficiency and capacity are noticeably affected.

A summary of all the foregoing indicates the following conclusions regarding the proper association of deep well pumps and wells:

If the pump proper cannot be set as deep as desired, use a long suction pipe not less than 20 feet long in order to minimize the air entrance.

Do not practice economy by selecting a small diameter well. Choose a well design incorporating a screen with openings that permit a copious supply of water at low velocity.

Locate the screen opposite sand formations or other water bearing formations so that the water will utilize all of the screen openings thereby reducing the entrance velocity. It is better for well performance if the water enters the screen slowly and rises in true artesian fashion up to the pumping level.

And finally, be sure the well contractor is familiar with the geology at the proposed well site.

It is hoped these remarks may reveal characteristics that heretofore have not been given enough consideration in selecting a ground water supply system of enduring satisfaction.



## Vertical Pump Applications

*By Jule H. Coffey*

**W**ITHIN the past few years the applications where deep well turbine pumps are used have increased very rapidly. This design found its first big field in irrigation in the far west. As the advantages became apparent, it was used for developing municipal and industrial water supplies. Such units are used for mine dewatering and sinking, highway underpasses, quarries, cargo handling, and other uses too numerous to mention. The turbine pump has been developed in small sizes to be used in a domestic system in the range of capacities between 1,000 and 3,000 gallons per hour. This fills the gap between the small domestic systems of the plunger and ejector types and the regular deep well turbine pump. This paper will stress other types of applications which are not so well known, but where the vertical pump has outstanding advantages.

The modern vertical deep well type turbine pump with its single or multistage pump units, short coupled to a vertical combination discharging and driving head, is continually finding new uses and in many cases replacing other types of pumping equipment.

The vertical pump has been recognized to have many advantages for use in a raw water river or lake pumping station (see fig. 1). In the first place, it is not necessary to have a dry well in which to install this equipment. The water is merely piped into the well and allowed to seek its own level. The well and pump are so designed that the pump bowls are submerged at all times. By using a wet well, it becomes unnecessary to use the extreme care required in water-proofing the concrete. Also, the use of a wet well entirely eliminates the danger of the well having any tendency to float during high water level. The well can be made relatively small in size, further reducing the cost. The motor and all electrical control apparatus and wiring

A paper presented on June 13, 1939 at the Atlantic City Convention by Jule H. Coffey, Vice-President, Pomona Pump Co., Pomona, California.

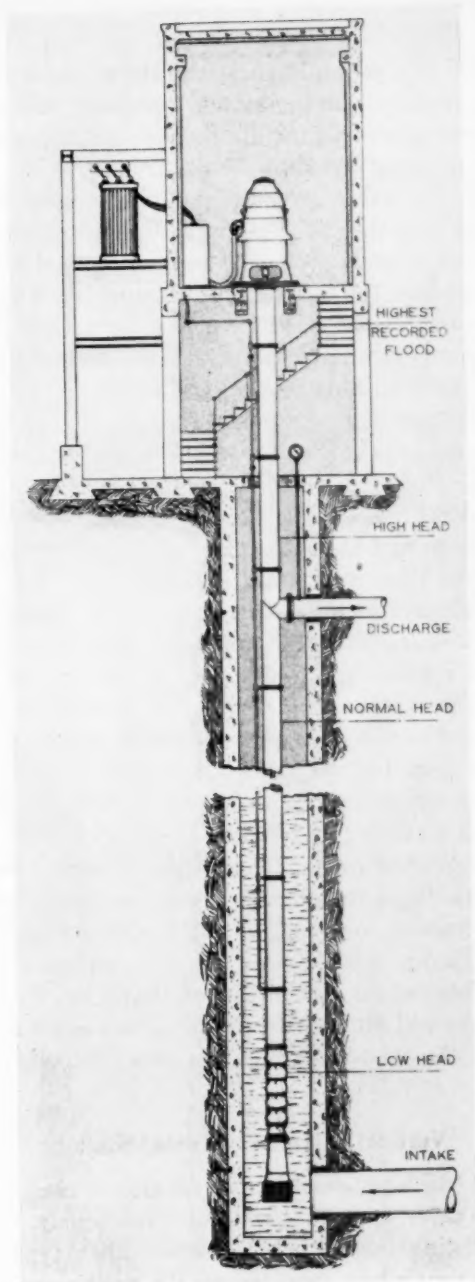


FIG. 1. Raw Water Pumping Station, Covington, Kentucky; 10,000,000 m.g.d., 390-foot head, driven by 800 h.p., 1,200 r.p.m. synchronous motor.

can be installed in a pump room at the top of the well; this greatly simplifies the problems of the station operator, and also keeps all of the electrical apparatus far above high water level and entirely out of danger of being flooded.

In general, this makes an ideal pumping station layout with a lower cost, and with the operating room up on top where everything needing any care is easily accessible and out of flood danger.

In addition to this, the turbine type of pump has a non-overloading horsepower characteristic, so that there is no necessity of equipping the pump with an extra large motor, and no danger of motor trouble due to overloads with fluctuating horsepowers.

The vertical type of pump lends itself readily to being designed with a semi-open impeller. This impeller has the lower vane edges machined in a cone shape, and is operated in a funnel shaped seat in the pump bowl. The close running clearance between the vane edge and the bowl seat takes the place of the close running clearance between wearing rings in the conventional enclosed impeller design. The advantage of the semi-open impeller in handling silt-laden, abrasive water is the fact that the close running clearance can be maintained by regulating the adjusting nut at the top of the motor which adjusts the vertical position of the pump shaft and impellers. Thus, when wear occurs it is not necessary to dismantle the pump to renew wearing rings, but it is necessary only to readjust the impellers from the pump operating room. This is a five-minute job.

A vertical raw water pump which is installed with sufficient discharge column to allow for the fluctuations in water level from season to season should be equipped with water lubricated bearings. It is the frequent practice to design oil lubricated pumps with a bypass port in the discharge bowl to allow the oil to escape. If oil can flow out from this port water can flow in, so that when the water level is high the oil tube will fill up with the silt-laden water and prove very destructive to the metal bearings operating directly on the metal drive shaft.

### **Vertical Pumps in Booster Stations**

The vertical turbine type of pump can also be used in many places for regular booster service quite advantageously. Sometimes to replace an existing installation of some other type, the vertical turbine booster may be connected to the existing suction and discharge lines, as a dry pit booster pump (see fig. 2).

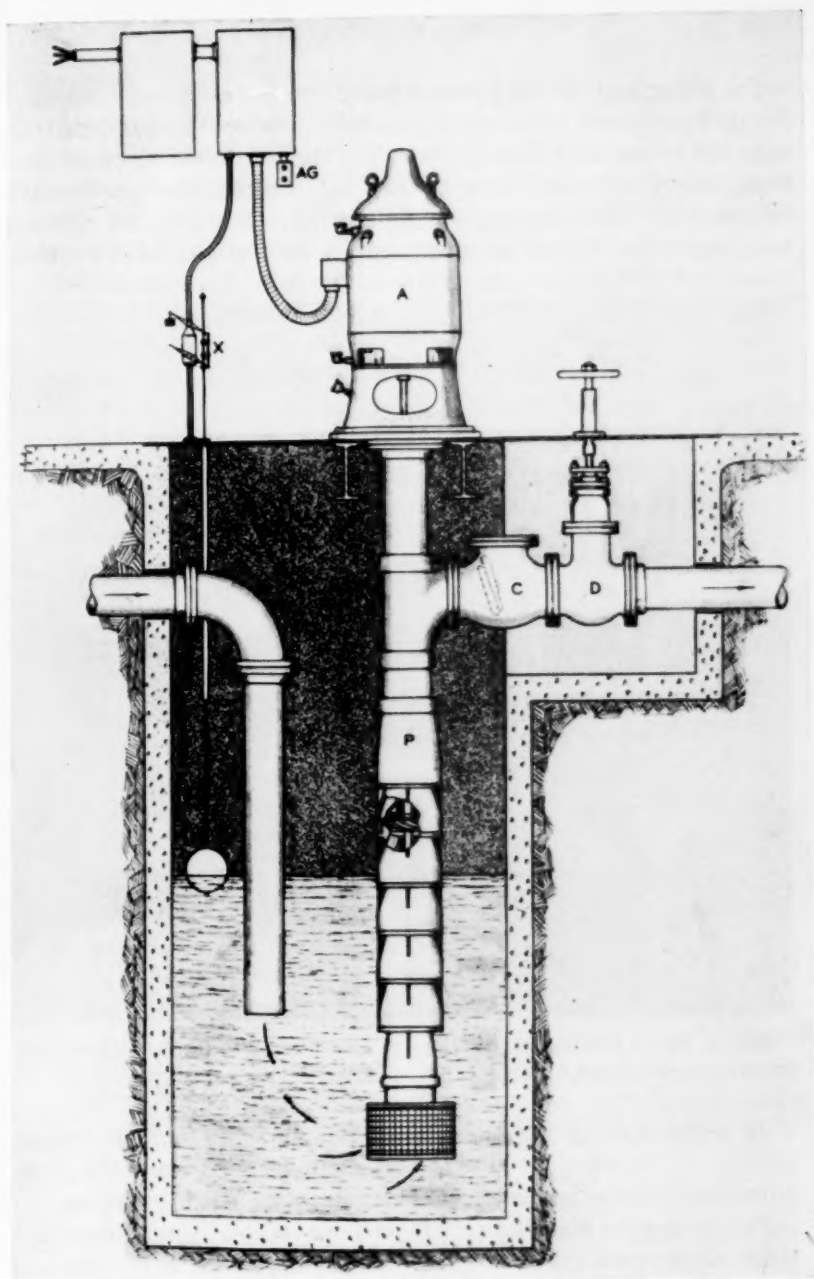


FIG. 2. Short Coupled Vertical Turbine Pump in Booster Service  
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For lifting and boosting from a sump, the necessity for a separate dry pit pump room is entirely eliminated. The vertical pump driving head can be installed directly above any sump with the short coupled pump bowls extending down into the water so that the impellers are submerged. The submerged pump entirely eliminates all priming troubles. Also, the submerged pump is not subject to cavitation.

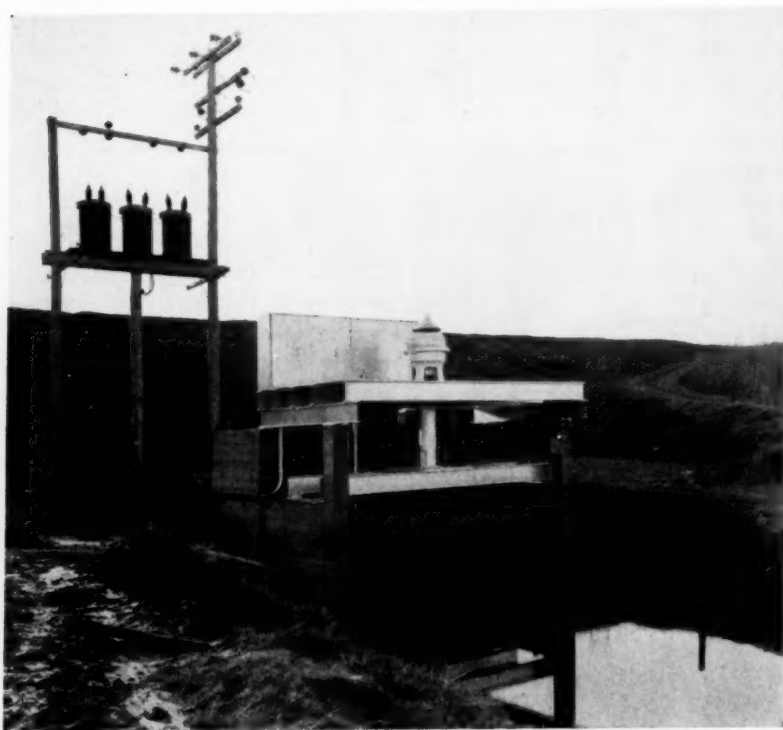


FIG. 3. Flood Water Drainage Pump, American River Flood Control District, Sacramento, California.

This destructive process is almost unknown in turbine pumps, because the impellers are always working under pressure. The water flows into the impeller instead of being drawn into it by vacuum.

Frequently the plant's piping layout can be greatly simplified with the vertical pump. The discharge of the pump can either be brought out above the foundation plate or at any point in the column pipe between the foundation plate and the pump bowls.

The vertical turbine pump has been developed to a very high degree of efficiency. When figured on a water-to-water basis the efficiency of the turbine pump will exceed other types in most cases, although the conventional methods of testing may make the turbine look poorer. In testing the horizontal pump, gages are placed on the discharge and the suction of the pump, so that in calculating the pump efficiency, the total dynamic head includes all of the entrance and suction pipe losses. In testing a turbine type of pump, the water-to-water method is used. In this it is customary to measure the pressure at the discharge fitting and the distance from that point to the water level in the well, thus, charging the pump with the entrance losses, suction pipe losses and discharge pipe losses to the discharge fitting. This difference in method of testing represents several points in efficiency.

The wet pit turbine type vertical pump lends itself admirably to pumping treated water into pressure mains. These pumps can be obtained in units of almost any capacity below 20 million gallons per day and staged for almost any required pressure. Here also the same advantages prevail: the pumping unit would be submerged in the delivery sump; no priming; no foot valve. This type of pump in the smaller sizes can also be used as a sump drainage pump to pump out the seepage in dry pits.

### **Large Capacities at Low and Medium Lifts**

In pumping large capacities at low and medium lifts, the trend has been very definitely toward vertical equipment (see fig. 3). In the extremely low lifts a straight propeller design is used. In most of the cases, however, a mixed flow design is most applicable, because it is adapted to a wider range and capable of pumping very large volumes at from ten to ninety feet head in single stage units. These designs lend themselves to the use of higher speeds than the large volute type pumps which have for many years been used in these applications. This makes the initial cost of the vertical equipment lower. Higher speed motors are not only less expensive but have better electrical characteristics.

Some of the applications where this type of equipment have found ready acceptance are in irrigation, drainage, flood control, sewerage and condenser water circulation. The advantages of the vertical low lift equipment are very much the same as the advantages stated above for the vertical turbine pump. The floor space requirements

are the very minimum; the installation of the vertical equipment over the sump eliminates the necessity for a dry pit pump room. All priming troubles are entirely eliminated. Cavitation which is always a problem in low lift high capacity equipment, is greatly reduced by submerging the pumping unit.

In the mixed flow design the impeller can be designed of the semi-open type which allows for vertical adjustment from the motor to compensate for any eventual wear.

The discharge fittings of the large capacity low lift pumps are fabricated by rolling and welding sections of sheet steel, allowing a large amount of flexibility in the design.

The center line of discharge can be brought out at almost any required point between the foundation plate and the pump bowls, thus greatly simplifying the piping arrangements in many instances. Also, the discharge can be brought out at the conventional 90-degree angle from the center line of the pump, or at any other special angle which may be required.

While it is true that the vertical pump is not the answer to all pumping problems, it has certainly given proof of having so much to offer that in considering any new pumping plants it will be well worth the engineer's time to consider vertical equipment.



## Fundamentals of Water Pretreatment

*By Frank Bachmann*

THE removal of impurities before filtration of water is an important step in water purification. The impurities may consist of color; turbidity in the form of clay, algae, microscopic organisms, dead organic matter or combinations of these; and bacteria. Tastes and odors are associated with products given off by microscopic organisms and certain industrial wastes. The elimination of tastes and odors by activated carbon and by pre- and post-chlorination with or without ammonia has previously been thoroughly discussed and consequently will not be mentioned further in this paper. Streeter's (1) studies on bacterial removals at several Ohio River and Great Lakes water treatment plants give definite conclusions on this phase of pretreatment. Therefore, this paper will be limited to a discussion of the removal of turbidity from water before filtration.

Many of the surface water supplies are periodically very turbid, especially those taken from western rivers such as the Missouri, Red, Colorado, Ohio, Mississippi and several others. Prior to selecting the exact treatment procedure for any of these water sources, it is desirable to make experimental sedimentation, particle size determination, and other tests based upon the considerable volume of knowledge on this subject, which is already available. Primarily it should be determined whether or not a turbid water requires the removal of the bulk of its suspended solids before further treatment, since it is well known that certain forms of turbidity adsorb coagulating chemicals without producing a proportionately beneficial effect. Tables showing the alum dosage required for various turbidities have been prepared at several treatment plants, based upon years of experience.

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A paper presented on June 13, 1939 at the Atlantic City Convention by Frank Bachmann, San. Sales Director, The Dorr Co., Inc., New York, N. Y.

Where very high turbidities were encountered, experience has shown that it was practically impossible to obtain a satisfactory water for filtration without excessive dosages of alum or other coagulants. Secondary coagulation and sedimentation were successfully used, where plant layout permitted. The plants on rivers with high turbidities persisting over a period of time, found it good practice to presettle the water before addition of coagulants, and in other cases a small dosage of chemicals to the raw water before presettling was beneficial. Water purification plants on the Ohio, Mississippi, Missouri and other rivers find presettling very effective. Examples of this practice are Louisville, St. Louis, Cincinnati, Covington, New Orleans, Kansas City, Omaha and Boulder City.

Fuller (2), as a result of his work on Ohio River water at Louisville, states that it is impractical to apply coagulants to a water containing large quantities of heavy suspended silt, which may be removed economically by plain sedimentation. He states further (3) that the presence of coarse particles which subside quickly causes a waste of chemicals in amounts equal to the quantities of original chemicals adsorbed plus a certain amount of aluminum hydrate, which becomes attached to their surfaces. The attached hydrate is thus removed before it coagulates to its full power the finer particles in the water.

The advantages of presedimentation in the treatment of highly turbid waters may be summarized as follows:

1. The furnishing of a water uniformly low in turbidity, thereby resulting in smoother plant operation;
2. Reduction in chemical costs, often as much as 70 per cent;
3. Less frequent cleaning of the coagulation basins, thus giving longer detention time in these basins.

Fleming (4) concluded from his softening experiments at St. Louis that very little lime and ferrous sulfate were wasted if the turbidity is less than 1,200 p.p.m. Presettling of the raw water before chemical dosing made material savings with higher turbidities. However, the higher turbidities must be of such a character that reduction will take place with presedimentation. He states that, if the coefficient of fineness is greater than 0.7 a water will, in most practical cases, settle without chemicals to a turbidity of 500 to 800 in from three to five hours' time. It is important, therefore, to make settling tests where possible and to determine the size of the suspended solids in the mud before concluding that presedimentation is practical. Other considerations are the cost of presettling tanks,

savings in chemicals, and the flexibility of the plant to take care of these periods of high turbidity.

The return of sludge to presedimentation tanks has been demonstrated by Fleming (5) as being very beneficial in accelerating the removals of suspended solids in raw water. At St. Louis, sludge from lime and ferrous sulfate treatment returned to the raw water increases the rate of sedimentation to the extent of 50 per cent. Final turbidities after two hours' detention are about equal to those obtained after three hours without the addition of treated water sludge.

As far as is known, no thorough study of the value of presedimentation in water purification has been made during the last decade to determine the economies of this phase of water treatment. The results of such an investigation would be an important contribution to the profession of water purification.

#### Factors in Flocculation

Flocculation or coagulation, may be defined as "a grouping together of finely divided suspended solids into small lumps." Experience has demonstrated that particles which settle individually quite readily, in the absence of appreciable convection currents from a disperse system, may be flocculated and induced to settle considerably faster as aggregates. Although a dispersion is considered as having stability, it is really an unstable system and is stable to the extent only that the electric charge on each particle repels other particles having similar charges. In order for flocculation to take place, however, the particles must collide. The probability of collision is, therefore, a determinative factor in the flocculation of a disperse system. Due to molecular energy within the disperse system and to mechanical motion induced by an outside energy source, the probability of collision between the particles of a system, whether of laboratory or commercial magnitude, is usually very great. It is obvious, with this condition of a system maintained, that when the coalescence of the particles does not follow immediately, some other factor is influencing the behavior of the system. The magnitude of this factor is the ratio of the number of collisions that lead to adherence to the total probable collisions, and is called the probability of adhesion.

The probability of collision and the probability of adhesion are, of course, resultants of more fundamental factors. The first is controlled by the molecular energy within the system supplemented

by mechanical motion induced by an outside energy source while the second factor is controlled by the intensity of the electrical charge of the particles plus the sphere of attraction surrounding each particle.

The function of electrolytes in the flocculation of a disperse system is to reduce the charge on the suspended particles until they are neutral or until they are sufficiently weak to permit flocculation under certain conditions. When the particles are completely neutralized, the process is said to take place in the "zone of rapid coagulation," while between this and a critical potential below which flocculation just begins the process takes place in the "zone of slow coagulation."

The factors covered in the above discussion may, on first glance, sound highly theoretical, but close examination with respect to experience in treating various types of disperse systems will show that the concepts of a probability of collision between particles and a probability of an adhesion for each collision are highly practical and susceptible of application. Electrolytes are added to increase the probability of adhesion to a point where flocculation may take place in either the zone of rapid coagulation or the zone of slow coagulation. The system may then be subjected to gentle agitation from an outside energy source to supplement and assist in promoting the probability of collision, the gentle agitation serving to greatly increase this probability. Also, the basis exists for explanation of the fact that certain disperse systems may be "mechanically" flocculated without addition of electrolytes. Such systems already exist in the zone of slow coagulation, probably near the critical potential, and agitation serves to aid "surfaces to find their mates."

This picture of the mechanism of the flocculation process is, of course, not accepted by all investigators who have studied and are studying the phenomenon. It does, however, constitute what may be termed a "practical theory" for the treatment of the suspensoid type of colloidal material. For more detailed information on the theory of flocculation and coagulation, reference may be made to Lasseter's work at the Mellon Institute (6).

### Flash Mixing

Some interesting observations with flocculation of various waters on laboratory and plant scale may be recorded. In regard to chemical dosing, it is the opinion of the writer that the chemicals should be

thoroughly and rapidly mixed with the water before coagulation takes place. In other words, quick mixing or flash mixing of the chemicals with the water prevents excessive reaction on a part of the water while the balance is underdosed. It has been demonstrated with lime and copperas treatment that, without flash mixing, the lime added has reacted to soften part of the water instead of reacting with the iron. This results in a waste of lime where softening is not desired.

After flash mixing, slow stirring to obtain collision and adhesion of the particles is the next step. The time required to obtain the best results from this slow-mixing or flocculation step depends on the amount of coagulant and the temperature, and less on the turbidity of the water. The older theory was that a fairly clear water cannot be flocculated. Waters with turbidities of under 5 are now being flocculated with low dosages of chemicals.

Clay, brick dust and other material have been added to water to increase the turbidity, particularly when most of it was due to diatoms or other forms of microscopic life. It was thought necessary to add these substances to give weight to the floc so the microscopic organisms would settle with the floc. When these conditions of low turbidity with high microscopic count prevail, short filter runs of 3 to 8 hours are not uncommon. Operators hesitate to use much alum because they state that they not only have the algae discharging on the filters, but the floc from the alum used. The writer has taken samples of the feed and overflow from the settling basins at plants when these conditions existed and found no difference between the two samples, indicating that at these times the settling basins were not functioning. About 0.3 grains per gallon alum was used at the time.

The way to handle a water of this character is to use a little more alum to obtain a greater volume of floc, which will entrain the turbidity and algae so as to settle this matter and discharge a clear water onto the filters.

#### **Removal of Algae and Microorganisms**

Four tests were made with Lake Michigan water at Winnetka, Illinois, by D. C. Reybold on June 7, 1935. The raw water had a turbidity of 5 p.p.m., and a temperature of 51°F. All flocculator tests consisted of 30 minutes flocculation followed by 30 minutes sedimentation. Microscopic counts were made on the raw water

and effluents of the four tests and reported as units per c.c. The dosage of alum used in these tests was 0.5; 0.75; 1.00 and 1.25 gr. of alum per gallon. Table 1 shows the reduction of organisms with the various dosages of alum to be from 68 per cent with 0.5 gr. alum to 94.7 per cent with 1.25 gr. alum.

Another similar series of tests for reduction in turbidity was made in John R. Baylis' laboratory in Chicago, June 20, 1933, on Lake

TABLE 1

*Flocculator Tests on Lake Michigan Water at Winnetka, Illinois, June 7, 1935*

MICROORGANISM VIEWED	RAW WATER	0.5 GR. ALUM	0.75 GR. ALUM	1.00 GR. ALUM	1.25 GR. ALUM
<i>Diatomaceae</i>					
<i>Asterionella</i> .....	335	125	70	50	15
<i>Nitzschia</i> .....	55	35	5	5	5
<i>Melosira</i> .....	5			5	
<i>Synedra</i> .....	20	15	5		
<i>Tabellaria</i> .....	15				
<i>Fragellara</i> .....	55				
<i>Diatoma Vulgara</i> .....	10				
<i>Chlorophyceae</i>					
<i>Protococcus</i> .....	200	55	35	25	20
<i>Scenedesmus</i> .....	5				
<i>Cyanophyceae</i>					
<i>Anabaena</i> .....	5				
<i>Clathrocytis</i> .....	5				
<i>Coelosphaerium</i> .....	5				
<i>Microcytis</i> .....	15	10	5		
<i>Protozoa</i>					
<i>Cryptomonea</i> .....	20				
<i>Uroglena</i> .....	15	5			
Total .....	765	245	120	85	40
Per Cent Removal .....		68	84.3	88.8	94.7

Michigan water using 0.5, 0.75, 1.0 and 1.5 gr. alum with 30 minutes flocculation and 30 minutes sedimentation. The results are shown in table 2.

Unfortunately, microscopic counts were not recorded in the Chicago tests, nor were turbidities on the settled water in the Winnetka tests. If, however, the per cent reductions of these two tests are examined, it will be noted that the turbidity and microscopic count reductions are quite similar with the same dosages of alum, except with that of 0.5 gr. which is an underdose in each case.

It is believed that with this type of water, where the turbidity of the raw water is made up primarily of microscopic organisms, the removal of turbidity is of the same order of magnitude as the removal of microscopic organisms. For instance, if 90 per cent of the turbidity is removed, 90 per cent of the organisms are removed.

The above results indicate definitely the necessity of proper dosage of coagulant, flocculation and sedimentation to relieve the filters of the load of microscopic organisms which shorten filter runs.

### Flocculation Time

Time of flocculation may be shortened at the sacrifice of low chemical cost. This is not economical, as additional mixing basin capacity is much cheaper than high consumption of chemicals. A definite relation between chemical dosage and mixing time has not been

TABLE 2

*Flocculation and Sedimentation Tests on Chicago Water Supply, June 20, 1933*

ALUM GR./GAL.	TURBIDITY		
	Raw	30 min. Flocculation 30 min. Settling	% Reduction
0.5	7.5	4.0	46.7
0.75	7.5	1.2	84.0
1.00	7.5	0.7	90.5
1.50	7.5	0.5	93.3

definitely established, but there are indications that on waters requiring low chemical dosage, doubling the dosage will cut the time in half.

The ideal chemical dosage is in the "zone of slow coagulation" as referred to above. This permits a material saving in the cost of chemicals but requires longer flocculation time, resulting in lower operating cost. Long periods of flocculation, say of one hour or more, improve the results over shorter periods, and there is no proof that very long periods are detrimental when optimum velocities are not exceeded or septic action of the circulation floc occurs. Longer periods of flocculation tend toward clearer water on the filters. Some operators feel that some floc should be carried onto the filter, but based on practical operation, most of them are of the opinion that the clearer the water the longer the filter runs, and the less troubles with sand filters due to cracks, mudballs, and other difficulties.

With mechanical flocculation it is the writer's opinion that to get the most out of any type of mechanism, the highest speed possible without breaking the floc is desired. If this is true, the maximum work possible is obtained from the mechanism. As stated previously, flash mixing of the chemicals is essential, followed by a constant slower speed to obtain flocculation. The ideal is to flocculate the water at the highest speed possible, this speed being at or slightly below the optimum for the floc under consideration. This method results in the production of pin-head size flocs, all of the same size which, when passed into the settling zone, settle at a uniform rate, leaving a clear line above. With varying size flocs, they would settle at different rates, the larger ones first, leaving the smaller ones in suspension.

After a satisfactory settleable floc is formed, it is essential that this floc be discharged into the settling tank without breaking it. High velocities are very detrimental and with alum floc the maximum velocity permissible is approximately 1.5 ft. per second. In water softening, the velocity can approach 2.0 ft. per second without ill effects on the settling characteristics of the floc.

Too much stress cannot be placed on the desirability of preparing water for filtration. If the flocculation step is adequate, settling will be efficient and the settled water will filter satisfactorily. The value of proper flocculation was not appreciated in municipal plants until Langelier (7) at Sacramento in 1921, and Smith (8) at Richmond, Virginia, in 1932, reported the results of their work on this subject. Fuller (2) (9), in his testing work at Cincinnati and Louisville as early as 1897, sensed the need of some such preparation step, but it was not until years afterwards that the water works profession had the means or equipment to accomplish the desired result.

### Mechanical Flocculation

The method which is in quite common use in this country and abroad to accomplish proper flocculation is the horizontal type of flocculator. This is used at some of the large water purification plants such as Milwaukee, Cincinnati, Toledo, Richmond and Denver, and numerous smaller plants. It consists of rows of paddles mounted on a horizontal shaft and driven by a gear motor through chain and sprockets or bevel gears located in a dry well. Experience has proven that gears or chain and sprocket drives operating directly in the water have relatively short life, hence the dry

well. A number of these paddle shafts are arranged either in parallel or transverse in the flocculation zone. They are rotated at the optimum speed indicated by the tests, which speed will be somewhere between 0.6 and 1.8 ft. per second, depending on the character of the raw water and coagulant used. The ratio of paddle diameter to depth of water, and the spacing of paddles are also factors in determining the speed of the paddles.

The term "coagulation" basin should be eliminated from the vocabulary of the water works profession, because today coagulation basins are not being constructed in water purification plants. The very old plants have coagulation basins where little or no mixing is provided and where the alum or other coagulant is dumped into the water and the water then passed on to detention basins, where the accomplishment of both coagulation and sedimentation was the hope of the designer. Today these steps are distinctly separate, and as a consequence we have a mixing or flocculation basin and a settling basin.

### **Effect of Flocculation on Detention Time**

The tendency today is to design settling basins with much shorter periods of detention than was the practice 25 years ago before the advent of mechanical mixing. Shorter detentions are possible because of better preparation of the floc for settling. It is quite common today to provide two hours detention in the settling basin with, and four hours without continuous sludge removal equipment. With a higher quality of water required by the public, particularly in regard to taste and odor, decomposing sludge should not be permitted to accumulate in the settling basins. This stale sludge aggravates the taste and odor problem.

As a result of this, continuous sludge removal equipment has been installed in a number of the newer water purification plants and it is being seriously considered in some of the older plants. Its advantages are:

1. The removal of the sludge with its taste producing properties,
2. Full detention available in the basin as sludge does not consume any volume of the tank,
3. Greater flexibility with greater capacity, as basin is not drained for sludge removal,
4. No spare units required to maintain settling capacity, and
5. Less noticeable pollution of the stream into which sludge is

discharged, because sludge is discharged continuously in small volume as against a large slug if discharged at intervals.

In concluding this discussion, it may be stated that with many types of water supplies, presedimentation should be practiced to remove the bulk of the suspended solids before coagulation. This saves chemicals and produces a more uniform water for the consumer. In connection with coagulation, it is essential to flocculate the very slow-settling, fine particles with chemicals and slow agitation to produce a rapid settling floc and clear water for the filters.

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**Discussion by Robert W. Furman.\*** Until recent years it has been customary to follow a somewhat conventional design in the construction of mixing, coagulation and sedimentation basins. The operation of those units was carried on in many instances without regard for economy or plant efficiency, while in many cases the design was such that large basin areas were ineffective and good operation was made difficult.

The ever increasing demand on the part of the public for water of better quality has had the desired effect on designing engineers and plant operators. In consequence, the recent developments and improvements have been rapid.

The process of preparing water by coagulation is, without question, one of the most important steps to consider. Much attention and

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\* Superintendent, Water Department, Toledo, Ohio.

study has been given to the proper mechanical functions of various types of equipment, with the result that a better quality of water has been produced at a much lessened cost.

At this time it seems to be a well established fact that regardless of the character of the raw supply or its tendency to vary, better operation has been possible through improved and properly designed mixing equipment. By these processes the tendency to induce adhesiveness has been perfected to the extent that its effect has been carried to every part of the succeeding stages of purification. The preparation of a dense and more granular floc that tends to resist fracture has assisted in more rapid and complete precipitation and has provided a suspension of such a character that prevention of floc penetration in filter sand has been corrected to a great extent.

One of the difficult types of raw water to treat is one which contains colloidal clay with the suspended particles covered with an oil film. Waters of this type are found in the drainage area of Northwestern Ohio, which territory is covered with oil fields. There is no question of the increased efficiency through the use of properly designed mixing equipment in treating waters of the above type.

Whether the repeated collision of particles has a tendency to break through the oily film and provide points for cohesion and adhesion, or whether each particle is blanketed by flocculated material is not known. It has been established, however, that more satisfactory results have been obtained than with plain coagulation and sedimentation. Turbidities of the type named above may not fall in the high range but are found frequently between 500 and 1,000 parts per million. They are smoky in appearance and show but little tendency to settle without the use of coagulants. No data have been found that would indicate the effect of presettling the oily turbidity type with a small dosage of chemicals. It is doubtful if the results would be very satisfactory under such conditions.

Under unusual circumstances the Maumee river, which supplies Toledo, Ohio, with its water, has required from 8 to 11 grains of aluminum sulfate for the reduction of turbidities ranging from 500 to 1,000 p.p.m. when this turbidity was finely divided and oil coated. Following the installation of mechanical mixing equipment, the reduction in coagulant has been between 15 and 25 per cent for the treatment of similar water.

Whether the improved reactions are caused by electrolytes or are induced by catalysts or result wholly or partially by physical means

does not alter the fact that mixing equipment of proper design and construction has offered a satisfactory method to assure stability, economy and better efficiency in the treatment of water.

If the proper flash mixing has been provided, it has been the experience at Toledo, Ohio, that the best operation of the mechanical mixing equipment has been obtained at a speed nearest the optimum when uniformly applied in each mixing stage. Variable rates of speed for the different stages of mixing, as well as too low speeds, have failed to produce as good results as were possible at uniform rates approximating one foot per second peripheral speed. This method of operation has had a tendency to prevent floc from settling in the mixing area when too slow speeds were applied as well as to eliminate the danger of fracture of floc from higher speeds.

Much work is being done in connection with the determination of the most desirable pH range to cover normal as well as peculiar situations and the relationship between temperature and the proper point for rapid coagulation. It has long been the writers opinion that applied water of low turbidity need not offer too great filter difficulties provided the suspended particles resist fracture, are dense, and are sufficiently large to prevent serious sand penetration. Modern mechanical mixing equipment has paved the way for many operating improvements.



## Coagulation of Soft Waters with Iron Salts

*By George E. Willcomb*

**T**HIS paper is an account of the experience obtained at Albany, N. Y., in selecting the coagulation process best adapted to the conditions imposed by the raw water. It deals with the efficiency of iron salts in the removal of color and manganese from a moderately soft water. The paper is developed in the following phases: (a) Definition of a soft water and its salient chemical characteristics; (b) Discussion of conditions influencing selection of a specific coagulant with special reference to color and manganese removal; (c) Results obtained with copperas and lime; (d) Results obtained with ferric sulfate; (e) Adsorption of manganese by a chemically coated sand.

The raw water discussed in this paper is typical of the impounded surface waters met with on the western watershed of the Hudson River extending from the New Jersey line to the Mohawk River. These waters lie just west of the limestone zone immediately adjacent to the water course and generally have a hardness well under 50 parts per million. The Catskill supply of New York City is typical of these waters.

The Albany supply consists of the impounded waters of Hannacroix and Basic Creeks stored in reservoirs of twelve and one billion gallons, respectively. Basic Reservoir discharges into Hannacroix through a three-quarter-mile-long tunnel. Before development these creeks had been analysed many times and nothing out of the ordinary had been found to prevent their use as a water supply.

The new supply was placed in service in December, 1932. The filter plant is located ten miles below the dam and consists of a raw water aerator, a mixing and conditioning process consisting of three

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A paper presented on June 13, 1939 at the Atlantic City Convention by George E. Willcomb, Supervising Chemist, Div. of Water and Sewage Treatment, Albany, N. Y.

units capable of being used in series or in parallel and providing a combined retention period of 32 minutes at a velocity of either 0.6 or 0.3 feet per second. The floc conditioned water is introduced into three coagulating basins of 3.2 hours combined retention period. The eight standard rapid filters have a combined capacity of 32 m.g.d. The 2 m.g. capacity pure well is baffled and serves as a reaction chamber for the lime (pH adjustment) process which maintains a pH of 8.5 in the water introduced into the mains. Experience has demonstrated that, for the local water, an average pH of 8.5 is the equilibrium point for the solubility of calcium carbonate.

Table 1 gives selected data on 52 fractional weekly samples of the raw water coming to the plant. From this table it will be noted

TABLE 1  
*Raw Water Data in p.p.m.*

	MAXIMUM	MINIMUM	AVERAGE
Turbidity.....	20	1	5
Color.....	66	11	21
Alkalinity.....	46	28	31
Hardness.....	39	27	32
Carbon dioxide.....	16	3	5
pH.....	7.7	6.5	7.0
Oxygen consumed.....	7.7	1.2	3.4
Dissolved oxygen.....	19.0	1.8	8.4
Manganese (Mn).....	3.3	0.2	0.5
Iron (Fe).....	0.9	0.05	0.2

that the color is moderate in amount, averaging 21 p.p.m. During the fall turnover of the storage reservoir in September the color reaches a maximum of 66 p.p.m., consisting of colloidal organic matter. This period of maximum color is of short duration, never exceeding two weeks. The maximum color for all of the other eleven months is only 36 p.p.m.

The hardness fluctuates between 39 and 27 p.p.m., with an average of 32 for the year. Practically all of the hardness is temporary. The carbon dioxide content averages 5 p.p.m. for the year and increases to 16 p.p.m. just prior to the fall turnover; at the same time hydrogen sulfide develops for the first time at the reservoir bottom and reaches a maximum concentration of 2.0 p.p.m. The aerators remove practically all of the hydrogen sulfide and by the end of the week follow-

ing the turnover, the hydrogen sulfide becomes completely dissipated throughout the reservoir. At this time the concentration of dissolved oxygen at the bottom becomes reduced to a minimum.

The interesting feature of the above table is the iron and manganese content of the raw water. As has been stated before, the preliminary analyses of the creek waters showed no manganese although outcroppings of low grade manganese ore were known to be scattered throughout the watershed. As has been the experience with other reservoirs in this vicinity, it is reasonable to believe that the manganese goes into solution as the result of the formation of humic acid and carbon dioxide evolved by bacterial action on the humus matter left in the soil when the reservoir bottom was stripped. These manganous salts probably exist as sulfides and carbonates.

TABLE 2  
*Manganese Content of Raw Water in p.p.m.*

	MAXIMUM	MINIMUM	AVERAGE
1933	0.20	0.04	0.06
1934	0.83	0.00	0.07
1935	1.14	0.00	0.09
1936	1.18	0.00	0.10
1937*	3.34	0.05	0.59
1938*	3.30	0.02	0.51

\* The figures for these years are greater due to the fact that the water was withdrawn continuously from the bottom outlet where the concentrations were greater.

It has been the experience at other reservoirs for the manganese to increase slowly in concentration and then gradually fall off. Such has been the case at Albany. Table 2 shows the manganese concentrations for the past six years. It would almost seem that the year 1937 was the peak year for manganese concentration and that a gradual recession might be expected from this point on.

It is quite evident from table 2 that manganese is the chief factor to be considered in the purification process although when the filters were designed little consideration was given this fact. In order to obtain a clearer idea as to the rôle played by manganese, table 3 showing the monthly concentrations is given. From these monthly data it will be noted that for seven months (January, February, March, April, May, November and December) the average man-

ganese concentration is only 0.10 p.p.m. with a maximum of 0.4 p.p.m. in one month only. In other words, the concentration for seven months out of the year is very low and quite uniform. For the remaining five months the concentration gradually increases during the summer, reaching a maximum just prior to the fall turnover.

When the plant started in 1932 manganese had not assumed the significance it eventually did, and little attention was at first given it in plant operation. Alum had been selected as the coagulant as a matter of course by the designing engineers, Whitman, Requaardt & Smith, and for four years this method of coagulation apparently worked well. As the water from the impounding reservoir could be withdrawn from four outlets, it became the practice to so manipulate

TABLE 3  
*Raw Water-Manganese Data*

	MAXIMUM	MINIMUM	AVERAGE
January—1938.....	0.20	0.04	0.09
February.....	0.10	0.04	0.08
March.....	0.20	0.06	0.12
April.....	0.20	0.01	0.11
May.....	0.40	0.05	0.14
June.....	0.50	0.02	0.26
July.....	1.50	0.29	0.88
August.....	3.30	1.18	3.12
September.....	2.80	0.08	0.80
October.....	1.40	0.08	0.33
November.....	0.20	0.05	0.14
December.....	0.20	0.03	0.15

these outlets as to assure a minimum amount of manganese in the raw water coming to the plant. Unfortunately, the optimum manganese content did not always coincide with the optimum algal content so that at times, in order to keep the manganese at a minimum by withdrawing the water from the top of the reservoir, this same water contained the maximum number of micro-organisms which resulted in filter runs as low as six hours.

Eventually, manipulation of the reservoir outlets failed to maintain a manganese concentration that could be handled by alum coagulation and the manganese in the filter effluent at times exceeded 0.5 p.p.m. resulting in an accumulation of manganese dioxide in the

mains that became a source of complaint and necessitated frequent hydrant blowoffs.

The first attempt actually to control the manganese by coagulation was made in October, 1936 when the point of application of the lime was changed from the pure well to the second pass of the mixers, the alum being introduced into the first pass as usual. By this procedure, the floc was formed in the first pass and was then subjected to a pH of from 8.0 to 9.0 during the last 20 minute period of mixing. Although this change resulted in the removal of the manganese, the aluminum hydrate was peptized to the extent that after-precipitation of the floc occurred in the mains and in the service connections of the consumers.

In the spring of 1937 the concentration of manganese in the mains became such a nuisance as to necessitate a change in the entire coagulation setup. It was therefore determined to change from alum coagulation to iron, at the same time adjusting the reaction pH in the mixer to 9.5 in order completely to precipitate the manganese. This decision was fortified by the excellent results obtained with iron coagulation at Providence and Baltimore, both treating somewhat similar waters (1).

### **Iron Coagulation Only in Summer**

From the preceding table of monthly manganese concentrations it will be noted that iron coagulation need only be practiced for the five summer months and that for the remaining seven months coagulation with alum would suffice. There was a definite advantage in this procedure for the reason that during the winter months the raw water could be coagulated with as low a dose as 0.3 g.p.g. which resulted in a decided saving when the corresponding low dose of lime was taken into consideration. It was believed that sufficient manganese dioxide would accumulate as a coating on the grains of filter sand during the period of iron coagulation as to act as a catalyzing agent for the removal of manganese during the winter months. This feature will be treated in the latter portion of this paper.

Four types of iron coagulation were considered: the original lime and iron process, chlorinated copperas, ferric chloride and ferric sulfate. In making the selection, comparative costs, nature of the reactions and conditions imposed by the raw water, and methods of handling and applying the chemicals under plant conditions,

naturally were considered. Inasmuch as all of the iron reactions were dependent upon the successful formation of a ferric hydrate floc and as during most of the year no interference with this floc formation need be expected from the raw water, a great latitude in the nature of the coagulant was offered.

In taking into account the various components of the raw water that might influence or complicate the reaction with iron salts, it was of course realized that no two waters are exactly alike or behave precisely similarly as respects coagulation. As Professor Charles Gilman Hyde (2) so aptly puts it: "All authorities agree that, among other things, floc formation with any coagulant is related to the hydrogen ion concentration in the water and that economic control is determined, at least in part, by the pH value. Other factors in coagulation are the electrical charges on the colloidal particles in the water, the character of the electrolyte, the extent and kind of buffer salts present, the amount and character of the particles causing turbidity, etc."

In reviewing the work of the following investigators: Miller (3), Bartow, Black and Sansbury (4), certain basic conclusions as to successful ferric floc formation are arrived at. Among them it was established that there should be present in a water a certain minimum quantity of ferric ion, that there must be present an ion of strong coagulating power such as the sulfate ion and that the hydron concentration must be properly adjusted. These investigators pretty well establish the fact that ferric coagulation can be successfully applied over a pH range extending from 3.8 to 10.0 or even higher although in some cases the floc forms slowly between pH 6.5 and 8.5. A very important fact brought out is—that low residual iron results from the use of ferric coagulants. The experience at Providence (1) with the three forms of iron coagulants indicates that they may be used interchangeably and that when compared on the basis of equal iron content there is less than 10 per cent difference in their respective performances.

### Use of Lime and Iron

After taking all things into consideration it was decided that for the initial work with iron salt coagulation the old lime and iron process was the most practicable. The following factors influenced this decision: the chemicals could be introduced without any plant changes whatsoever; the chemical costs were favorable, being even

lower than alum coagulation; and finally, conditions of the raw water were favorable, that is, the color was not too high nor was it of a colloidal nature bearing electrical charges combative to those borne by the ferric hydrions.

The regular plant storage bins were of 12 tons capacity each and were located in a dry place so that the ferrous sulfate would not lump up to any extent. The regular plant dry feeders could eject the iron solution either into the aerator influent or effluent. The three 10-minute units comprising the mixing and floc conditioning system each possessed initial two minute high velocity mixing compartments so that, if need be, three chemicals might be introduced at the same time, flash mixed and floc conditioned for ten minutes.

Although it would have been possible to attempt to oxidize the iron in the aerator without the addition of an alkali, this procedure was not attempted. Instead, it was decided to introduce the copperas into the first pass of the mixer following aeration of the raw water. By this means the bicarbonate of iron initially formed was neutralized by the lime introduced in the second compartment and the pH raised to 9.5. At the end of the conditioning period a most satisfactory ferric floc was introduced into the coagulating basins.

There was one unfavorable feature connected with the use of copperas. Oddly enough, a scarcity developed in the supply of this commodity which had always been regarded as a waste product. A sudden demand by pigment manufacturers made carload shipments impossible and spot consignments by truck became the rule. This condition was one of the decisive factors leading to the selection of ferric sulfate the following year.

Coagulation with copperas and lime was employed from August 1 to October 15, 1937. Satisfactory flocculation was secured with an average of 1.09 g.p.g. copperas and 1.01 g.p.g. calcium hydrate. The average manganese removed amounted to 99 per cent, the average residual iron in the effluent was 0.12 p.p.m. and the average color of the effluent was 12 with a maximum of 36 p.p.m. The color of the effluent remained under 12 p.p.m. all through the summer and up to the time of the fall turnover of the reservoir on September 4. At this time the manganese in the raw water reached 3.34 p.p.m. and the hydrogen sulfide soared to a maximum of 2.0 p.p.m. resulting in an apparent color of 104 in the raw.

It was soon realized that conditions obtaining in the reservoir at the time of the fall turnover imposed new complications as to coagu-

lation which were beyond the capacity of iron salts to overcome. The conclusion was reached that iron salts from which ferric hydroxide would normally be precipitated were not adapted for the removal of color when organic acids tending to produce colloidal iron compounds were in the ascendancy in the raw water. Due to high color and residual iron in the effluent and a distinct bitter taste which could not be removed by carbon, iron coagulation was abandoned on October 15, 1937.

A detrimental feature connected with the use of copperas was a distinct lowering of the palatability of the final product. Shortly after starting the process the plant analyst detected a slight change in the taste of the water which was difficult to describe. At the same time complaints of a slight bitter taste in the water were received from consumers. The lessening of the palatability was not great enough to call for corrective measures; however, intermittent complaints persisted throughout the summer and only ceased when the iron coagulation was discontinued and alum substituted in its place. That this condition was caused by the copperas is unmistakable, yet it is difficult to ascribe the chemical reactions involved. Inasmuch as the iron content of the effluent was appreciable in amount during all this period, it is probable that this was the cause.

### Use of Ferric Sulfate

On the advent of the season of high manganese concentration in the spring of 1938 it was decided to use ferric sulfate as a coagulant. This iron salt dissolves readily under controlled conditions and upon dissociation reacts with the natural alkalinity of a water or with added lime and forms a ferric hydrate floc which is insoluble throughout a pH range from 3.5 to 10.0. This fact is especially important in the removal of color in alkaline waters of pH value over 9.0 where at the same time it is necessary to oxidize and coagulate manganese which can only be accomplished at a pH of from 9.0 to 9.5.

The precipitation of manganese is a complicated process involving both chemical and physical action. In the natural formation of soluble manganese compounds at the bottom of a reservoir, bicarbonates and sulfides predominate. These are manganous forms and are not readily oxidized by the dissolved oxygen in the water alone. However, when a strong alkali is present the oxidation is quickly brought about. The mechanics of the action appears to take place in two ways: first, the oxidation products are adsorbed by the ferric

hydrate floc and, secondly, since there is always present some manganous hydroxide, this substance also adsorbs colloidal manganese particles although most of the adsorption must of necessity be accomplished by the ferric hydrate flocs which predominate.

### **Related Experiences in Other Cities**

The phenomenon of the adsorption of colloidal manganese by manganous hydrate flocs suggests the use of this agency for the removal of manganese when it forms its characteristic coatings on the sand grains of a rapid filter handling a water of high manganese content. This is closely related to the aeration-contact process for the removal of iron and manganese as employed at Memphis, Tenn., and Lincoln, Neb. More will be said about this catalytic action later in the paper.

Experience at Providence indicated that ferric sulfate could be successfully used as a coagulant for soft moderately colored waters and plans were made for its use in the spring of 1938. The product used was Ferrisul. Great improvements had been made in the physical qualities and its price had been reduced so that the cost of the process was less than that of lime and iron.

The product was of rice size and came in 175-pound bags. It was easily handled by the mechanical conveyors and stored without caking in the overhead bins. The regular dry feed machines discharged into special lead-lined baffled dissolving pots furnished by the chemical manufacturer. These machines were agitated by an electric stirrer and the water admitted to the dissolving pot was measured by a Rotometer, an ingenious device for measuring low flows. The water was constantly adjusted to a ratio of 2 to 1 for the Ferrisul. The dissolved product discharged into a special Durion ejector which forced the solution through a 2-inch rubber hose to the first pass of the rapid mixer. No trouble at all was experienced in dissolving the iron and corrosion of the apparatus was negligible. The lime was applied at the inlet to the second mixing unit ten minutes after the iron was added. After a 20-minute conditioning period a large, well separated floc had been formed.

Ferric sulfate coagulation was employed from May 13 to October 8, 1938. It was discontinued on account of coagulation difficulties brought about by the interfering action of organic compounds brought into suspension following the fall turnover of the reservoir. These difficulties were quite comparable to those encountered during the

reservoir turnover of the previous year when the lime and iron process had to be discontinued. During the period of ferric iron coagulation the average color of the effluent was 4 p.p.m. and the manganese content 0.02 p.p.m. This type of treatment was satisfactory in every way and in addition, proved to be the cheapest thus far employed. One outstanding feature was the fact that at no time did it affect the palatability of the water in the slightest degree.

Table 4 gives a comparison of the results obtained by the two iron processes. It gives cost data per million gallons of water filtered. It also gives maximum, minimum and average figures in parts per million for the chemical components and the efficiencies of manganese

TABLE 4  
*Results Obtained with Copperas as Compared with Ferrisul*

COPPERAS				FERRISUL			
	Max.	Min.	Ave.		Max.	Min.	Ave.
Copperas, g.p.g.....	1.80	0.56	1.09	Ferrisul, g.p.g.....	1.30	0.43	0.71
Calcium hydrate, g.p.g.....	1.30	0.50	1.01	Calcium hydrate, g.p.g.....	1.42	0.32	0.77
Cost, copperas per m.g.....	—	—	1.83	Cost, ferrisul per m.g.....	—	—	1.37
Cost, lime per m.g.....	—	—	0.93	Cost, lime per m.g.....	—	—	0.73
Cost, wash water per m.g.....	—	—	0.73	Cost, wash water per m.g.....	—	—	0.54
Total cost, per m.g.....	—	—	3.49	Total cost, per m.g.....	—	—	2.62
Reaction, pH in mixer.....	9.9	7.0	8.5	Reaction, pH in mixer.....	9.7	7.1	8.5
Manganese in raw, p.p.m.....	3.34	0.23	1.96	Manganese in raw, p.p.m.....	3.27	0.08	1.05
Manganese in pure, p.p.m.....	0.15	0.00	0.03	Manganese in pure, p.p.m.....	0.10	0.00	0.02
Iron as Fe in raw, p.p.m.....	0.96	0.08	0.35	Iron as Fe in raw, p.p.m.....	0.90	0.06	0.33
Iron as Fe in pure, p.p.m.....	0.42	0.02	0.12	Iron as Fe in pure, p.p.m.....	0.08	0.01	0.03
Color in raw, p.p.m.....	104	23	41	Color in raw, p.p.m.....	66	12	25
Color in pure, p.p.m.....	36	7	12	Color in pure, p.p.m.....	14	1	4
% Reduction Mn, basins.....	65	10	44	% Reduction Mn, basins.....	100	6	43
% Reduction Mn, filters.....	100	84	96	% Reduction Mn, filters.....	100	74	95
Overall % Mn, reduction.....	100	93	99	Overall % Mn, reduction.....	100	77	97

removal. The cost data are based on the actual cost of the chemical as delivered at the unloading platform at the plant. Wash water is figured at the actual cost of producing one million gallons of filtered effluent. It includes pumping, chemical and labor charges connected with producing and storing the water in the elevated wash water tanks.

The base costs are as follows: copperas, \$1.16 per 100 pounds; Ferrisul, \$1.40; calcium hydrate, \$0.64; and wash water \$15.02 per million gallons. Wash water costs are included for the reason that any improvement in coagulation is reflected in longer filter runs resulting in lower wash water costs.

In comparing the cost data it will be noted that with Ferrisul all chemical and wash water costs were lower, resulting in an average net saving of \$0.87 per million gallons. It will be observed that the iron and manganese factors of the raw water were fairly equal for the two periods although in 1937 the average manganese content of the raw water averaged considerably higher. The reason for this is that copperas was used for 79 days while Ferrisul was used 148 days. Copperas was started late in the season on August 1, while Ferrisul started on May 13. It will therefore be realized that the period during which Ferrisul was used includes a large portion of the spring season in which the manganese content of the raw water is always low.

The manganese in the effluents for the two periods was practically the same. There was a big difference in the residual iron content of the effluents: for 1937 the iron residual averaged 0.12 p.p.m. while in 1938 it averaged only 0.03 p.p.m. In the same way during 1937 the color of the effluent averaged 12 p.p.m. while in 1938 it averaged only 4.

#### **Efficiency of Manganese Removal**

The efficiencies of manganese removal are divided into three sections: the actual removal effected by the basins, the amount actually removed by the filters based on the amount actually coming onto them in the basin effluent and the total overall efficiencies based on manganese in raw and pure well. It will be observed that the removal of manganese effected by the basins was low, averaging about 43 per cent for both years. During both periods the filters removed 95 per cent of the manganese coming onto them and the overall efficiencies were 99 per cent and 97 per cent respectively. From these figures it will be seen that considerably more than one half the original manganese was stored up in the filter sand during the run. It can be readily understood that, if washing conditions were not perfect, an increasing residual of manganese would permanently become stored in the filter sand and gravel in the form of a coating inclosing the particles.

The inability of the basins to remove more than 43 per cent of the manganese is certainly a disquieting feature. One of the principal arguments advanced favoring the use of iron salts over alum is the comparatively greater specific gravity of iron floe. The basins being of definite size, the only way that more floe can be removed is to increase its weight. A possible solution might be the use of sodium

silicate added with the coagulant with the idea of producing larger and quicker settling flocs. As has been said before, the retention period of the local basins is 3.2 hours. When this is compared with a retention period of 48 hours as at Providence it can well be appreciated why the results obtained at one plant cannot be compared with those of another.

### Adsorption of Manganese by Sand

The coating of the sand grains with manganous oxides is not without its advantages, however. It makes possible the removal of the small amount of manganese present in the raw water during the winter months by the regular alum coagulation. The removal of manganese by zeolites and special contact filters has long been standard practice. In the case of contact beds the medium may be pyrolusite (manganese dioxide), coke, or sand coated with the oxides of manganese artificially. It has been found that a regulated well coated sand is as effective as a medium composed of pyrolusite itself. In both cases, successful operation is dependent upon the efficiency of the back wash.

It will of course be understood that in order for a silica sand to be effective in the removal of manganese the sand grains must first be coated by aging. A natural corollary is—that the coating must not be allowed to become too thick else the initial loss of head in the filters will increase to the extent that the lengths of runs are materially decreased. That these conditions are formidable, goes without saying; either the efficiency of manganese removal is too low or the filter runs become too short.

At the local plant the low retention period of the coagulating basins (3.2 hours) afforded an excellent means of coating the sand grains with the necessary amount of manganous oxide. Following the first summer's run with iron coagulation the grains had become adequately coated. An analysis of the sand following the second summer's run disclosed that the coating on the grains amounted to 23.4 lb. of manganese as Mn and 11.4 lb. of iron as Fe per ton of dry sand. This is equivalent to 1.17 per cent and 0.57 per cent by weight. Such a heavy coating naturally would increase the frictional resistance of the sand and it is not surprising that the initial loss of head increased on an average of 0.5 feet. This resulted in shortening the filter runs fully 30 per cent.

It was expected that during the winter period of coagulation with

alum and the accompanying presence of carbon dioxide, an appreciable amount of manganese would be dissolved from the sand. On March 1, 1939 representative samples of sand were collected from all eight filters and the amount of manganese ranged from 13.9 to 31.8 lb. per ton of dry sand, an average of 20.9 lb. for all beds. It is thus apparent that only 11 per cent of the manganese had been washed from the sand during the winter period of coagulation with alum. These results would lead to the conclusion that ultimately the manganese coating would have to be removed by an acid solvent. Experiments are now being conducted to ascertain the most efficient solvent.

It is interesting to note just how much may be accomplished in the actual removal of manganese by adsorption engendered by a gradual

TABLE 5  
*Manganese in Raw Water and Pure Well during Period of Manganese Removal by Adsorption*

	RAW WATER			PURE WELL		
	Max.	Min.	Ave.	Max.	Min.	Ave.
November 1938.....	0.20	0.05	0.15	0.06	0.02	0.03
December 1938.....	0.20	0.03	0.15	0.18	0.01	0.07
January 1939.....	0.22	0.12	0.17	0.08	0.03	0.05
February 1939.....	0.28	0.18	0.23	0.13	0.08	0.10
Average.....			0.17			0.06

building up of a chemical coating on the sand grains. Table 5 gives the manganese data for the four months following the shutting down of iron coagulation. The alum dosage during these four months averaged only 0.35 g.p.g.

A matter of interest in the above table is the gradual increase of the maximum and average manganese content of the raw water as the winter progressed. With a dosage of only 0.35 g.p.g. of alum and a reaction pH of 7.0 in the mixing basin the manganese content of the filtered effluent averaged only 0.06 p.p.m. which is equivalent to an average reduction of 65 per cent for the period. From these data it is clearly evident that under local conditions the phenomenon of adsorption of colloidal manganese compounds by an adequately coated sand is definitely established and may be relied upon to care for the manganese contained in the raw water up to a definite maxi-

mum, say 0.5 p.p.m. When this maximum is reached the effluent would contain approximately 0.15 p.p.m. which is about as much as a distribution system will stand without complaints eventually being received from the consumers as to discolored water in the services.

### Summary

1. Soft waters (under 50 p.p.m.), moderately colored, readily react with the four common forms of iron coagulants, providing the raw water contains no interfering organic colloids. With impounded supplies the turnover period is a critical one, in that organic colloids are carried into suspension and prevent proper floc formation, resulting in high color and residual iron in the effluent.

2. Impounded supplies on the west side of the Hudson River and south of the Mohawk are likely to show increasingly high concentrations in manganese during the first ten years of their use. These concentrations are uniformly low during the late fall, winter and early spring seasons, but increase steadily during the summer, attaining a maximum during the latter part of August just prior to the fall turnover.

3. The lime and iron process of coagulation adequately removes manganese at a pH of from 9.0 to 9.5. With local waters of low turbidity and color the amount of these components remaining in the effluent is too high. Due to low concentrations of mineralization and organic matter the Albany raw water is quite susceptible to taste provoking compounds. Thus, in the case of the lime and iron process the residual iron in the effluent imparts a slight but distinct bitter taste to the water.

4. Coagulation with ferric sulfate effectively removes manganese at a pH of from 9.0 to 9.5, and produces an effluent of very low color and residual iron content. As in the lime and iron process, colloidal organic matter brought into suspension by the fall turnover seriously interferes with the efficiency of the process. The palatability of the water is in no way impaired and the method is cheaper than that of lime and iron.

5. Due to the low manganese removal effected by the basins (43 per cent reduction) the filter sand becomes heavily coated with manganous oxides. This condition makes it possible to adsorb 65 per cent of the manganese in the raw water during the winter period of coagulation using a very low dose of alum.

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**Discussion by Elwood L. Bean.\*** The paper presented by Mr. Willcomb is one that should interest all operators of plants in the soft water regions. It should lead them to compare in their own plants the efficiency and results with ferric coagulants against the commonly used alum.

The paper should likewise interest engineers for here once again has been demonstrated the value of ferric coagulants and of equal interest to the engineer is the demonstration of the desirability of a plant design which will provide the greatest flexibility possible in operation. Too often a certain coagulant has been selected "as a matter of course" and a plant has been designed similar to some existing plant just because the location was in the same valley or the same neighborhood and therefore the water might be the same. Experience has shown that study of streams preliminary to reservoir construction is not even sufficient to predict five years in advance. Construction of a plant incorporating flexibility may be somewhat more costly but it is good insurance.

It is well known that organic acids are produced by decomposition at the bottom of deep reservoirs. That manganese may be dissolved by these acids is likewise well established. Therefore since manganese is a common ingredient of soils it seems that its presence in waters from deep reservoirs must be anticipated.

We are fortunate that manganese can be removed by such a simple process as the iron coagulation at higher pH. In this process the superiority of the ferric form over the ferrous form has been demonstrated by the author thus confirming results published from the city of Providence (1, 2, 3, 4), treating a water similar in most respects but containing even less hardness and alkalinity. (Analyses

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\* Of New England Water, Light & Power Associates, Cranston, R. I.

of the effluent of these two plants when using ferrous coagulant show surprising likeness.) Of interest is the observation of the author that the ferric method is the "cheapest thus far employed" and the figures of \$2.10 for chemicals using the ferric against \$2.76 per m.g. with the ferrous. Experimentally and in plant operation it has been demonstrated (2) that the dosage required in p.p.m. is inversely proportional to the ionic charges on the iron, or that the dosage of ferric iron required is two-thirds the dosage of ferrous iron. Calculation from this basis will always indicate the comparative cost of the coagulants. The amounts of alkali required must also be taken into account in any calculation of total cost. The superior effluent with ferric coagulant, however, should practically eliminate ferrous coagulant from consideration for soft waters.

In the application of ferric coagulant a coating of sand grains by manganese dioxide and iron has been noted by the author which he fears may make sand cleaning necessary. In one plant where iron coagulant was used the coating after eight months was about 1.0 per cent, while after four years, or six times as long, the coating had not reached 1.5 per cent. I believe that, provided the wash rate is adjusted to the most effective point, a similar result will be found which would make cleaning unnecessary for prolonged periods. Failure to wash filters properly can, of course, in a period of a few weeks even make cleaning a necessity in any plant.

The plan to utilize this same manganese coating as a catalyzing agent for removal of manganese, while using alum coagulant during the winter months, is one which is subject to question. To the consumer one of the cardinal virtues for a water supply is constancy of quality. This is particularly true of industrial users such as high pressure boiler operators, dyeing industries, laundries, breweries, electroplaters, and various manufacturing and canning industries. When changing coagulants as suggested wide changes are bound to occur unless the effluent is lime treated to a pH in excess of 9.0. If this is done then even 0.10 p.p.m. of manganese is sufficient to cause troubles.

Abandonment of the ferric coagulant in the fall of the year, as stated in this paper, may appear to stand as an indictment against the process. To this writer, whose experience with iron coagulants includes more than five years continuous operation, however, the picture appears much differently. There appears to be a great deal that we do not know about the reactions of coagulation of different

waters. There is yet an opportunity for a great deal of research along these lines. We know, however, that these various organic compounds are destroyed by pre-chlorination and that their power of interference passes so that proper coagulation may be obtained. Whether there are other methods less expensive or more satisfactory only the research and experiences of the future will tell.

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**Discussion by A. H. Ullrich.\*** Mr. Willcomb's rather complete résumé of his experiences with iron coagulation at Albany practically limits this discussion to experiences with the same type treatment at Fort Smith. Chemically the new Lake Fort Smith water is similar to the Albany water, as described by Mr. Willcomb, with the exception that it contains no manganese. The total hardness varies from 10 to 30 p.p.m., the methyl orange alkalinity varies from 8 to 20 p.p.m. and the pH varies from 6.8 to 7.3. Turbidities encountered so far have ranged from 2.0 p.p.m. to 410 p.p.m. To date there has been no excessive color but there have been periods of considerable algae concentration.

The corrosive nature of the raw lake water makes it imperative that the treatment used include corrective treatment for anti-corrosion as well as clarification, sterilization and taste and odor removal. Unfortunately the design of the clear well for the new plant is such as to make control of any treatment given the filtered water exceedingly difficult and this fact practically necessitates the combination of the corrective treatment with coagulation.

The plant design contemplated the use of either alum or ferric chloride, the equipment for making the latter by the Scott-Darcey process being included in the original setup. As operation is of necessity intermittent, the production and use of our own ferric

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\* Chemical Engineer, Water Department, Fort Smith, Arkansas.

chloride was ruled out and alum was used until studies of iron coagulants other than ferric chloride could be made.

During the period when alum was used exclusively, an attempt was made to increase the pH somewhat in the coagulation process but this was quickly discontinued when it was found that a pH of approximately 7.5 caused a considerable reduction in the size of the floc. It was then decided to coagulate with alum at a pH of 7.2 to 7.3 until further studies of other coagulants could be made.

As the Fort Smith water comes to calcium carbonate stability at pH of about 8.5 and as it was not desired to increase the pH in the treated water much above this point, the use of straight copperas and lime was not considered. Chlorinated copperas was considered, however, and actual plant tests lasting several weeks were made. While effective treatment at pH 8.5 was obtained the coagulant cost

TABLE 1  
*Comparison of Treating Costs*

OCTOBER (ALUM)		NOVEMBER (FERRISUL)	
Alum.....	\$1.277	Ferrisul.....	\$1.316
Lime.....	.215	Lime.....	.449
Chlorine.....	.708	Chlorine.....	.340
Ammonia.....	.376	Ammonia.....	.184
Total.....	\$2.576	Total.....	\$2.289

was considerably higher than when alum was used and it was decided to try ferric sulfate.

Several plant tests using ferric sulfate (Ferrisul) were first made and the results were so promising that more effective dissolving equipment was installed and continuous use was started on November 1, 1937. While the delivered cost of Ferrisul at Fort Smith is high, it is more effective with the Fort Smith water than alum. Also, we have found that its oxidizing action reduces the chlorine and ammonia consumption when, as in Fort Smith, the water is given the chlorine-ammonia treatment in the mixing chamber. Table 1 gives a comparison of the chemical treating cost per million gallons for October, 1937, when alum was used and November, 1937, when Ferrisul was used. The raw water was almost identical for the two months.

The Ferrisul therefore showed an actual saving of \$0.287 per mil-

lion gallons even though the cost of the additional lime required to raise the pH from 7.3 with alum to 8.5 with Ferrisul was charged against it.

While the use of ferric sulfate in a combination coagulation and corrective anti-corrosion treatment has been very effective, a number of interesting side issues have developed. First, for economical use it is imperative that suitable dissolving equipment be provided. In this discussor's opinion, the complete solution of either iron or alum before introduction into the water to be treated is important but this importance is increased in the case of ferric sulfate. Likewise the point of application with respect to the point of application of lime should be given serious thought.

Another interesting side issue concerns the relative penetration of dirty filters by iron and alum, especially when such filters are overloaded due to lack of wash water. The Fort Smith experience indicates that under these conditions both will penetrate but that alum penetration may continue longer without detection. The easy detection of iron penetration may actually hasten correction of the fault. In Fort Smith it has been an added factor in causing the installation of the Baylis surface washing system for mud ball removal.

The upward pH adjustment necessary for effective anti-corrosion treatment has led to two additional observations which may be of interest. First, as Todd reported in the January, 1938, issue of the *Journal of the A.W.W.A.*, rapid increase of the pH in distribution systems containing tuberculated mains may cause some of the tuberculation to break down and disperse in the water. Such dispersion of iron oxide must not be confused with red water caused by active corrosion nor must it be confused with iron penetration through the filters. In our case it required no less than four months to bring the water in all of the distribution system to the desired pH equilibrium point and a "wave" or red water seemed to precede the attainment of the corrective pH.

The second and more important observation relative to the upward pH adjustment has been the rather rapid removal of zinc from galvanized pipe at pH 8.5. At this pH the zinc forms a mushy substance which has the appearance of zinc hydroxide. This sloughs off easily and may cause localized complaints of "gray" water with "white" sediment after customers have installed new galvanized service lines. While we have not made dissolved zinc determinations on

such water we do not believe that much, if any, is carried in solution. However, the rate of removal has alarmed us to the extent that we now plan a series of service tests to determine the economic feasibility of using galvanized pipe if we continue corrective treatment at pH 8.5.

The subject of the action of high pH waters on zinc linings is new to this discussor but experience with a soft water at pH 8.5 causes him to wonder if such waters treated to a higher pH may not actually dissolve and carry zinc in solution. After reading an advance copy of Mr. Willcomb's paper this discussor wondered if the slightly bitter taste obtained when using copperas might not have been caused by dissolved zinc rather than from iron since at Albany a higher pH was carried in the mixer while using copperas than with Ferrisul. In a separate communication Mr. Willcomb kindly furnished additional data showing that "in 1937 when copperas was being used the average pH was 8.7 while in 1938 when Ferrisul was being used the pH of the pure well water was 8.3." The higher pH in 1937 together with the description of the taste as slightly bitter would lead one to suspect a small amount of zinc in solution. However, Mr. Willcomb states that the taste was found in samples taken from the pure well which, as far as he knows, had not come in contact with any galvanized piping. This, as he says, seems to preclude the assumption that zinc caused the taste.

As stated before, the action of high pH waters on zinc linings is a new subject to this discussor and additional information along this line will be welcomed.



## Iodide Technic for Colorimetric Determination Of Chlorine in Water

By F. W. Gilcreas and F. J. Hallinan

**D**URING many years of general routine use, the ortho-tolidine test has been demonstrated to be of value in the control of chlorination. Unfortunately, however, the reagent is not specific for chlorine but reacts with many compounds commonly found in water supplies. Interference by these substances generally imposes on the color resulting from the reaction with chlorine a similar color that cannot be differentiated, and a falsely high chlorine content is therefore indicated. The magnitude of the effect of interfering agents such as ferric iron, manganese, and nitrites has been studied by many investigators (1), but none of the methods suggested for eliminating or compensating for such interference has proved completely satisfactory.

The presence of manganese, particularly in stored surface waters, to an extent that gives serious interference in the ortho-tolidine test, and the need of a method for the accurate determination of residual chlorine in swimming pools where nitrites may frequently be present in amounts greater than 0.1 p.p.m., make it imperative that a method be developed to measure chlorine residuals accurately in the presence of these substances. In the eighth edition of *Standard Methods for Water Analysis* (1), it is recommended that the starch-iodide method in neutral solution be used to measure chlorine in the presence of more than 0.3 p.p.m. of nitrites or 0.01 p.p.m. of manganese. The procedure employs a technic that is difficult, if not impossible, to use in the field. Furthermore, Hallinan and Thompson (2) have demonstrated that accurate determinations can be made only by control of hydrogen ion and potassium iodide concentrations within narrow limits, and

A paper presented on June 12, 1939 at the Atlantic City Convention by F. W. Gilcreas and F. J. Hallinan, Division of Laboratories & Research, New York State Department of Health, Albany.

that no combination of these concentrations suitable for the thio-sulfate titration appears to eliminate interference by organic or other chlorine adsorption compounds of probable low bactericidal activity. However, if a system of accurately measuring the iodine liberated by the action of chlorine on potassium iodide could be developed for use at relatively high pH values, such interfering effects might be reduced.

It is well known that iodine develops specific colors with a number of adsorption indicators that are more sensitive than starch. Of these,  $\alpha$ -naphthoflavon has been recommended for use as a titration indicator (3); it is available as a crystalline substance stable in alcoholic solution for long periods when protected from daylight. It seemed desirable, therefore, to determine whether it could be used in a colorimetric test for the iodine liberated by the reaction of free chlorine in water on potassium iodide, and thus if a method applicable to the routine control of the chlorination process could be developed.

The sensitivity of adsorption indicators diminishes as the ratio to the material adsorbed increases. Thus only as much indicator should be used as would be saturated by the maximum amount of iodine to be measured. It was found that the color of the  $\alpha$ -naphthoflavon indicator at saturation with iodine was blue, and any further increase in iodine concentration tended to develop a green color. When the indicator was in excess, the blue color diminished in intensity with decreasing iodine concentrations, or in the presence of excess potassium iodide changed in color through blue-red to red, brown, and then yellow, the color of the indicator alone in water. In a range of chlorine concentrations from 0.1 to 1.0 p.p.m., the maximum change of color was observed when approximately 20 p.p.m. of indicator and 20 grams per liter of potassium iodide were added. Amounts of indicator greater than 20 p.p.m. impart an excessive turbidity to the sample and should be avoided. When  $\alpha$ -naphthoflavon is added to a solution of iodine the maximum color reaction is practically instantaneous, except that with amounts of iodine near the limit of sensitivity of the indicator the rate decreases and a period of three minutes is required for complete color development. The color compound does not exist in solution but is a colloidal suspension stable for from five to fifteen minutes if not agitated.

According to the nomogram of Hallinan and Thompson (2), with potassium iodide concentration of 20 grams per liter, the pH value

should be 6.7 or greater to eliminate interference by oxidized manganese with the chlorine-potassium-iodide reaction. Because of the

TABLE 1

*Effect of pH values on elimination of interference with the  $\alpha$ -naphthoflavon test for residual chlorine*

SAMPLE	COLOR*	TURBIDITY*	OXYGEN CONSUMED*	MAN-GA-NESE*	NI-TRITES*	FERRIC IRON*	MEAN R†			RANGE OF ORTHO-TOLIDINE CHLORINE VALUES USED TO DETERMINE R†
							$\alpha$ -Naphthoflavon test with pH 6.7	$\alpha$ -Naphthoflavon test with pH 8.9	$\alpha$ -Naphthoflavon test with pH 10	
			p. p. m.	p. p. m.	p. p. m.	p. p. m.				p. p. m.
Surface supply with manganese, chlorinated	0	25	2.7	1.1	0.03	2.5	0.30	0.31	0.26	1.0, 1.5
Surface supply with manganese, not chlorinated	0	25	2.7	1.1	0.03	2.5	0.0	0.0	0.0	0.9, 1.1
Normal surface supply; (1)	0	20	2.1	0.02‡	0.002	0.66	1.09	0.96	0.80	0.1, 0.95
	(2)	6	5	2.3	0.01‡	0.001	1.01	0.93	0.85	0.19, 0.75
Heavily polluted river water, chlorinated	25	20	7.4	0.038‡	0.005	0.7	1.40	1.05	0.59	0.23, 0.60
Sewage, Imhoff tank effluent chlorinated	55	190	170.0	0.17	0.002	2.0	18.4	3.70	1.80	0.03, 0.40

\* *Standard Methods for the Examination of Water and Sewage*, 8th ed., 1936, 309 pp. Tests made before chlorination.

† R is the apparent chlorine value indicated by the  $\alpha$ -naphthoflavon test at the given pH divided by the ortho-tolidine estimate of chlorine concentration. The values of R in five observations made at varying residual chlorine values are averaged. The double cell technic to compensate for color and turbidity of the sample was used in all tests.

‡ This manganese was present as manganese dioxide, and interference with the ortho-tolidine test was compensated for by subtracting the false residual of the raw water.

possibility that chlorine adsorption compounds might also react with potassium iodide and produce an interference, as suggested by Hallinan and Thompson, it seemed desirable to determine the effect

of a more alkaline medium on the sensitivity of the  $\alpha$ -naphthoflavon test, as well as the relationship between chlorine values obtained by this technic and by the standard ortho-tolidine procedure when applied to different types of natural waters. Chlorine concentrations were estimated by comparison with standards prepared from chlorine solutions of known concentration in distilled water.

TABLE 2  
*Effect of pH on color development with the  $\alpha$ -naphthoflavon test for residual chlorine*

RANGE OF CHLORINE CONCENTRATIONS	NUMBER OF OBSERVATIONS	MEAN RATIO*	MIDRANGE $\alpha$ -NAPHTHO-FLAVON VALUES AT pH 6.7	CORRESPONDING $\alpha$ -NAPHTHO-FLAVON VALUE AT pH GIVEN	pH OF $\alpha$ -NAPHTHOFLAVON TEST
			<i>p.p.m. chlorine</i>	<i>p.p.m. chlorine</i>	
0.0-0.3	1	1.0	0.15	0.15	4.5
0.3-0.6	1	1.0	0.45	0.45	4.5
0.6-1.2	1	1.0	0.90	0.90	4.5
0.0-0.3	5	1.37	0.15	0.11	8.2
0.0-0.3	11	1.42	0.15	0.11	8.9
0.3-0.6	4	1.06	0.45	0.43	8.9
0.6-1.2	3	1.02	0.90	0.88	8.9
0.0-0.3	4	1.55	0.15	0.10	10.2
0.3-0.6	1	1.06	0.45	0.42	10.2
0.6-1.2	1	1.10	0.90	0.82	10.2
0.0-0.3	1	2.00	0.15	0.08	12.0
0.3-0.6	1	1.17	0.45	0.38	12.0

\* Mean ratio is the p.p.m. chlorine with  $\alpha$ -naphthoflavon at pH 6.7  $\div$  p.p.m. chlorine with  $\alpha$ -naphthoflavon at pH given, when standards are buffered at pH 6.7.

The results of these experiments, tables 1 and 2, indicate satisfactory elimination of manganese interference at pH 6.7, but higher chlorine values with the  $\alpha$ -naphthoflavon test than with the ortho-tolidine technic in waters of high turbidity and organic content. This latter interference appears to diminish with increasing pH values. If a sensitivity of 0.1 p.p.m. is accepted as satisfactory for the control of the chlorination process, it would appear from this study that at pH 8.9 this sensitivity could be attained, and interference by chlorine

adsorption compounds would not cause a significant discrepancy between the results with the  $\alpha$ -naphthoflavon and the ortho-tolidine reactions when applied to potable waters.

To adjust the hydrogen ion and potassium iodide concentrations as indicated, a dry mixture of sodium potassium tartrate, potassium bitartrate, sodium diethylbarbiturate, and potassium iodide in the correct proportions was found to exert no chlorine demand and also to prevent precipitation of calcium and magnesium salts from hard waters.

In the development of a routine technic using this indicator, it seemed desirable to use 2-ounce French square bottles containing 50 ml. of the chlorinated water, since this is the type of equipment frequently employed with the standard ortho-tolidine test. A calibrated measuring spoon for dispensing the buffer-iodide mixture and a standardized medicine dropper for measuring the indicator appeared the simplest equipment for use in the field.

Solutions containing 0, 0.05, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.8, 1.0, and 3.0 p.p.m. of chlorine were prepared in these bottles and to each was added the correct amount of buffer salt and potassium iodide to produce pH 8.9, and potassium iodide concentration of from 18 to 20 grams per liter. Each solution was then treated with 20 p.p.m. of  $\alpha$ -naphthoflavon indicator as an alcoholic solution and the resulting colors noted. A sharp differentiation in reaction with the varying amounts of chlorine occurred, the color varying from pale straw through orange, red, violet, and blue to green with increasing concentrations.

As a result of this preliminary investigation, a technic for the iodometric determination of free chlorine in water using  $\alpha$ -naphthoflavon as an indicator has been developed as follows:

### Reagents

#### 1. Buffer-iodide mixture

Sodium diethylbarbiturate..... 1.0 part

Sodium potassium tartrate..... 0.1875 part

Potassium acid tartrate..... 0.0625 part

Potassium iodide..... 1.25 part

1.8 to 2.0 grams of the above mixture dissolved in 50 ml. of distilled water should adjust the pH to 8.9.

#### 2. A 0.2-per-cent solution of $\alpha$ -naphthoflavon in 95-per-cent ethyl alcohol.

### Procedure

To a 50 ml. sample warmed to 10°C. or higher if necessary, add 1.8 to 2.0 grams buffer iodide mixture, with a calibrated spoon; dissolve, add 0.5 ml. indicator, and mix. Let stand without further stirring for three minutes. Compare in tilted black box designed to hold 2-ounce French square bottles in two 3-bottle rows. Rest a ground glass plate on sample and standards. For double-cell procedure, be sure that sample and controls are on side near light source. This should be north light, reflected from white paper or unglazed milk glass.

Permanent color standards essential for any routine technic have been developed by matching with mixtures of inorganic colored solutions the colors obtained by the addition of the indicator to chlorine solutions treated as described. Red, yellow, blue, and gray acid solutions of inorganic salts were prepared and blended in the proportions shown in table 3 to give colors corresponding to the given chlorine concentrations. These standards have been found stable for approximately four months when exposed to direct sunlight, and for twelve months when stored in the dark.

To compensate for the lack of colloidal suspension in the permanent standards, it is necessary to view the test portion and the standard through a ground glass plate. As indicated, the adsorption compound coagulates on long standing or shaking and thus it is necessary that the test portion be allowed to stand undisturbed for 3 minutes to provide for full color development under all conditions of iodine concentration.

To determine the effect of interfering substances, particularly manganese and nitrites, portions of tap water containing increasing amounts of each were prepared and tested according to the established technic. The results are given in table 4. With nitrites, ferric iron, or oxidized manganese, no reaction occurred in concentrations that would be found in the average water supply.

Chlorinated waters representing distilled water, potable waters, polluted waters, and sewage have been examined for residual chlorine using the  $\alpha$ -naphthoflavon technic as outlined, and the standard ortho-tolidine test as modified to minimize nitrite interference (1). Table 5 shows that in waters low in organic matter, such as distilled water and potable waters, the results of the two technics were in close agreement. With turbid waters of high organic content as

shown by the oxygen consumed value, there was a decided tendency for the  $\alpha$ -naphthoflavon test to yield higher chlorine values than the ortho-tolidine reaction.

TABLE 3  
*Preparation of permanent color standards  $\alpha$ -naphthoflavon test for residual chlorine*

CHLORINE	COLOR SOLUTION			
	Gray*	Red†	Yellow‡	Blue§
<i>p.p.m.</i>	<i>ml.</i>	<i>ml.</i>	<i>ml.</i>	<i>ml.</i>
0.05	0.34	0.15	9.0	0.0
0.10	0.58	0.27	10.5	0.0
0.15	0.77	0.40	12.0	0.0
0.20	0.97	0.54	13.5	0.0
0.25	1.14	0.71	15.0	0.0
0.30	1.28	0.90	16.5	0.0
0.40	1.48	1.28	19.5	0.0
0.50	1.76	1.60	22.6	0.0
0.60	2.24	1.81	25.6	0.0
0.70	3.34	1.89	23.7	0.0
0.80	4.92	1.85	3.0	0.0
0.90	5.58	1.65	0.0	7.1
1.0	5.98	1.42	0.0	9.9

The amounts of color solutions given above are combined and diluted to 50 ml. with distilled water.

\* Cobalt chloride ( $\text{CoCl}_2 \cdot 6\text{H}_2\text{O}$ ) 31.9 grams; copper sulfate ( $\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$ ) 67.5 grams; nickel sulfate ( $\text{NiSO}_4 \cdot 6\text{H}_2\text{O}$ ) 75.0 grams; concentrated hydrochloric acid, 32 ml.; concentrated sulfuric acid 45 ml.; dissolved in distilled water and diluted to 500 ml.

† Cobalt chloride ( $\text{CoCl}_2 \cdot 6\text{H}_2\text{O}$ ) 476 grams dissolved in distilled water and filtered. 100 ml. concentrated hydrochloric acid added and diluted to 1 liter.

‡ Potassium platonic chloride ( $\text{K}_4\text{PtCl}_6$ ) 2.0 grams; copper sulfate ( $\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$ ) 3.75 grams; concentrated sulfuric acid ( $\text{H}_2\text{SO}_4$ ) 2.5 ml.; concentrated hydrochloric acid 100 ml.; dissolved in distilled water and diluted to 1 liter.

§ Copper sulfate crystals ( $\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$ ) 150 grams and 100 ml. of concentrated sulfuric acid, dissolved in distilled water and diluted to 1 liter.

In sewage, results with the new technic were even more at variance with the ortho-tolidine results and fluctuated with the reaction-time allowed. In the examination of sewage it seems impossible to determine the ratio of residual chlorine to chlorine adsorption compounds by any of the established methods. Because of these irreg-

ular reactions the technic as at present developed is probably of questionable value for determining residual chlorine in sewage. In the examination of potable waters, however, it appears to give precise results and to eliminate interference by manganese and nitrites.

Temperature variations did not cause serious error in the residual chlorine value, except that at temperatures less than 10°C. off-shades were produced that made color comparison with permanent standards difficult. As with many colorimetric procedures, differences in lighting produce off-shades; since the standards were developed for use in natural daylight only, artificial light cannot be used.

TABLE 4

*Effect of interfering substances in chlorine-free tap water on the  $\alpha$ -naphthoflavon technic*

MANGANESE	NITRITES	FERRIC IRON	APPARENT RESIDUAL CHLORINE	
			Ortho-tolidine	$\alpha$ -naphthoflavon
p.p.m.	p.p.m.	p.p.m.	p.p.m.	p.p.m.
0.0	0.0	1.0	0.0	0.0
0.0	0.0	10.0	0.02	0.0
0.0	0.0	20.0	0.05	0.0
0.05	0.0	0.0	0.05	0.0
0.1	0.0	0.0	0.10	0.0
1.0	0.0	0.0	1.1	0.0
5.0	0.0	0.0	>2.0	0.0
0.0	0.1	0.0	0.01	0.0
0.0	1.0	0.0	0.06	0.0
0.0	10.0	0.0	0.15	0.0

If a constant source of artificial light was available, standards suitable for such conditions could readily be developed. Interference by color and turbidity of the sample was found to be approximately the same as with the ortho-tolidine test and can be compensated for by use of the usual double cell technic.

The method outlined has had limited use by operators of filtration plants and engineers in the field; they have reported satisfactory results as soon as familiarity with the technic and the color comparison is attained. The abrupt change in shade of color with increasing concentrations of chlorine has been found useful because of the definite qualitative indication of change in chlorine concentration.

TABLE 5

Comparison of residual chlorine values by the ortho-tolidine and  $\alpha$ -naphthoflavon tests

RELATIVE CHLORINE VALUES TEN MINUTES AFTER CHLORINATION							DESCRIPTION OF SAMPLES CHLORINATED		
Waters examined		Range of chlorine concentrations, $\alpha$ -naphthoflavon test	Number of observations	Mean R*	Midrange $\alpha$ -naphthoflavon value	Corresponding ortho-tolidine value	Type of water	Oxygen consumed	
Type	Number							p.p.m.	Turbidity
Distilled	1	0.0-0.3	9	0.99	0.15	0.15			
		0.3-0.6	5	0.91	0.45	0.48			
		0.6-1.2	3	1.04	0.90	0.87			
Surface	3	0.0-0.3	11	0.94	0.15	0.16	Impounding reservoir	2.1	20.0
		0.3-0.6	5	0.85	0.45	0.53	Impounding reservoir	2.3	5.0
		0.6-1.2	6	1.15	0.90	0.78	Polluted river	7.4	20.0
Well	3	0.0-0.3	6	1.17	0.15	0.13	Drilled well	0.7	<5.0
		0.3-0.6	5	1.07	0.45	0.42	Drilled well	0.2	<5.0
		0.6-1.2	4	1.05	0.90	0.86	Dug well	1.6	20.0
Swimming pool	1	0.0-0.3	2	1.19	0.15	0.13		3.7	5.0
		0.3-0.6	2	1.18	0.45	0.38			
		0.6-1.2	6	1.06	0.90	0.85			
Sea water	1	0.0-0.3	5	1.11	0.15	0.14		5.5	15.0
		0.3-0.6	1	1.17	0.45	0.38			
		0.6-1.2	3	1.19	0.90	0.76			
Diluted sewage	1	0.0-0.3	2	1.61	0.15	0.09		36.8	50.0
		0.3-0.6	1	1.06	0.45	0.43			
		0.6-1.2	4	1.10	0.90	0.82			

\* R is the apparent chlorine value indicated by the  $\alpha$ -naphthoflavon test at the given pH divided by the ortho-tolidine estimate of chlorine concentration. The values of R in the number of observations given made at varying residual chlorine values are averaged. The double cell technic to compensate for color and turbidity of the sample was used in all tests.

### Summary

A colorimetric test not affected by the presence of manganese, nitrites, and similar interfering substances has been developed for the determination of free chlorine in water. It is based on the liberation of iodine from potassium iodide by chlorine in a test portion buffered to a pH value of 8.9 by potassium bitartrate, sodium potassium tartrate, and sodium diethylbarbiturate, and the use of  $\alpha$ -naphthoflavon as an adsorption indicator. Color standards have been prepared from inorganic colored solutions. The test is sensitive to 0.1 p.p.m. of chlorine and eliminates interference by 0.5 p.p.m. of manganese and 5 p.p.m. of nitrites and ferric iron.

Chlorine adsorption products react with this technic and higher residual chlorine values are obtained than with the standard ortho-tolidine test in the examination of turbid waters and those with large concentrations of organic matter. The method is thus of limited value in the examination of sewage or highly polluted waters.

The technic is more complicated than the ortho-tolidine test and, therefore, will probably not replace it in cases where interference by manganese and nitrites is not a factor. It has, however, proved valuable in routine use for the accurate determination of residual chlorine in the presence of amounts of these interfering substances that preclude the use of the ortho-tolidine test.

### References

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**Discussion by George E. Willcomb.\*** This method for the determination of chlorine excess has been used by the Albany Filters for three years, carrying out the suggestion made by Mr. Gilcreas and using apparatus supplied by his laboratory. Manganese in the Albany raw water ranges from 0.1 part per million in the winter to 3.5 p.p.m. just prior to the fall turnover of the impounding reservoirs. At times, during periods of faulty coagulation the concentration of manganese in the filtered effluent has reached a maximum of 0.5 p.p.m. giving rise to many inaccuracies in the determination of the

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chlorine excess. Resort was made to the hydrogen peroxide modification of determining the excess but this was not entirely satisfactory under local conditions and resulted in generally low readings. The sodium flavon indicator has proved to be a valuable tool and has given consistent results.

As Mr. Gilcreas has described, the field method consists of adjusting the pH by means of adding a tablet consisting of 0.45 gram each of Rochelle Salts and potassium iodide to 50 c.c. of the water. These manufactured tablets greatly facilitate operations in the field. The naphthoflavon reagent is added by means of a dropper and after the time interval the unknown is compared with standards contained in ordinary two-ounce bottles viewed through a ground glass mat. Good matches are obtained in the color comparisons by the operator after a little experience and readings up to 0.5 p.p.m. excess easily obtained.

During winter conditions with a cold raw water and manganese concentrations averaging 0.24 p.p.m. ortho-tolidine reagent added to such a water will give an apparent chlorine reading of 0.11 p.p.m. whereas with the same water the naphthoflavon indicator will give no reading at all. At all ranges of manganese concentration from 0.1 to 3.5 p.p.m., in both warm and cold raw waters, naphthoflavon has consistently given zero readings showing that within such limits the indicator is unaffected by manganese.

The following daily routine manganese and chlorine excess data are interesting and typify what can be accomplished by the indicator at a time when manganese interference is at a maximum. The manganese concentrations at different parts of the plant were as follows:

Raw—2.39, Basin Effluent—1.40, Filtered—0.06, Pure—0.06. Ortho-tolidine added to these waters gave the following readings: Raw—0.50, Basin—0.15, Filtered—0.04, Pure—0.20. The readings with naphthoflavon were: Raw—0.00, pure—0.15. In other words, the naphthoflavon was not influenced by the manganese in the pure well water and gave nearly a true reading for the chlorine excess commensurate with dose of chlorine actually added to the pure well water.



## Illinois's Experiences in Lime Softening With Short-Time Upward-Flow Clarification

*By H. A. Spafford and C. W. Klassen*

ILLINOIS, as many other states, has practically reached the saturation point in public water supplies. Relatively few municipalities now remain without such supplies which have population sufficient to finance soundly such an improvement even with outside financial assistance. Thus the future major activities of the water works profession will probably be applied toward improving the quality of existing supplies.

The safe sanitary quality of a supply is now generally taken for granted by the water-consuming public. In addition to safe quality there is a noticeably increasing interest in the other desirable attributes of a water supply and there is a public demand for waters of satisfactory mineral character purely from the standpoint of ordinary domestic consumption.

The mineral quality of a public water supply has public health significance. Experiences have shown that a supply physically and minerally objectionable because of high iron content, hardness, excessive chlorides, dissolved gases in objectionable amounts, etc., results in the use of many private wells, cisterns or springs in lieu of using the safe public supply. The majority of these private sources are subject to contamination. Many such unsafe private sources are often cross-connected in the homes to the public water supply. The Illinois legislature recognized this fact and has granted authority to the State Department of Public Health for the control of mineral quality of public water supplies in addition to sanitary quality and adequacy.

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A paper presented on June 13, 1939 at the Atlantic City Convention by H. A. Spafford, Assistant Sanitary Engineer, and C. W. Klassen, Chief Sanitary Engineer, Illinois Department of Public Health, Springfield, Ill.

While Illinois now has many water treatment works for improving mineral quality, including practically all of the types commonly used in lime-soda or zeolite processes, nevertheless scores of municipalities still need such improvements. Thus the writers being engaged in public health work have deemed it expedient to give considerable study to water treatment problems and especially to improved methods of treatment which seem to have merit and may either decrease installation or operation costs.

There is a continual effort by the water works profession to produce water of improved sanitary and mineral character at lower cost. Decreased installation costs as well as operating economy are the goals to be reached, while retaining or improving the quality of the product produced. Improvements in mixing and coagulation have made it possible to reduce retention periods during clarification, from many hours or days, in some of the older treatment plants, to only a fraction of the time formerly required. During the last few years some plants have been constructed in which the total time for mixing and clarification has been reduced to approximately one hour. Because several of these short-time lime-softening plants are located in Illinois, including, we believe, the first municipal installation employing modern mass precipitation, the writers have had opportunity to observe operations frequently and assemble data on results produced.

### Basic Principles

It appears to us that mass precipitation as applied by Charles Spaulding in the "Precipitator" and Green and Behrman in the "Accelerator" involve two basic principles when used for lime softening. *First*, the water treated with chemicals and super-saturated is brought into intimate contact with previously precipitated sludge particles, thereby exposing the super-saturated water to a tremendous area of previously formed crystal surface, tending to produce, more rapidly and completely, the precipitation of the minerals producing hardness in water. *Second*, clarification is effected by mechanical means bordering upon filtration or straining of the water as it passes through the sludge. Small, newly formed particles of precipitate tend to be entrapped by the larger and heavier sludge particles, resulting in very rapid production of low turbidity effluents without resorting to plain sedimentation.

Recognition of the value of contact with previously formed precipi-

tates in lime softening of water is not new. Sludge return in some conventional plants has been practiced for many years, and increased efficiencies often obtained. However, the amount of sludge returned has usually been limited by the ordinary methods of mixing and clarification, so that the optimum exposure of crystal surface was either very difficult or impossible. Sludge return beyond a certain point has often resulted in clarification difficulties and, with surface waters, return of sludge containing unstable organic material frequently created serious taste and odor problems. The two short-time upward-flow treatment processes, which we propose to discuss, were developed with the object of obtaining maximum benefit from contact with previously formed precipitates, without the usual limitations and difficulties ordinarily experienced with plain sludge return.

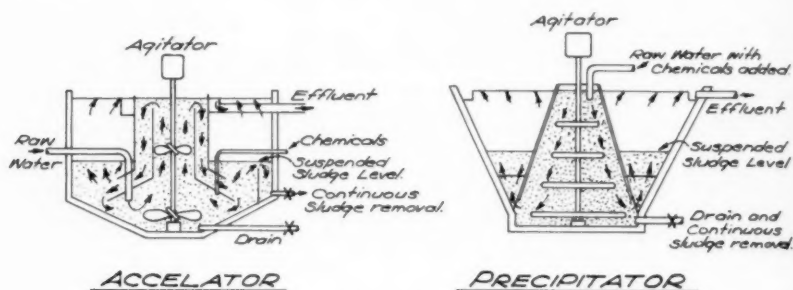


FIG. 1. Schematic Design of Accelerator and Precipitator

Figure 1 shows the schematic design of both the Accelerator and the Precipitator. In the Accelerator raw water is introduced into the central draft tube at one side near the bottom, while the chemicals are applied on the opposite side. Propellor type agitators mix the raw water and chemicals with the existing sludge particles. The travel is upward through the central tube, thence downward through the annular space between the central and outer tube. As the mixture of sludge and water issues between the skirts of the two tubes a large portion is pulled back into the bottom of the central tube and is thus recirculated. Clarified water issues upward from the sludge blanket between the outer tube and the basin wall and passes to the top of the basin for removal over weirs or launders. The sludge throughout the mixing and clarification zones is held in suspension by the force of agitation and although the sludge particles are rela-

tively close together, there is free circulation of water around the precipitates, thus permitting exposure of the treated water to the crystal surfaces of the particles.

The draft tubes forming the mixing compartment are fabricated from steel plates. The basins may be either square or circular in plan and can be constructed of steel plate or reinforced concrete. The larger units are usually of concrete construction.

For treatment in the Precipitator, raw water has usually been dosed with chemicals in a dosing well before coming into the central cone at the top, although one plant in Illinois introduces the raw water and chemical separately into the mixing compartment. Mixing of the water, chemicals and sludge is effected by a vertical shaft agitator equipped with several horizontal arms. Flow is from the top of the inner cone downward, thence upward through the sludge blanket in the annular space between inner and outer cones. Transfer of rotary motion from the inner to the outer cone is prevented by several radial fins placed in the bottom of the annular space between the cones. This detail of construction is clearly shown in Mr. Jensen's photograph given in his paper on "Water Softening at Minneapolis" and published last year (*Jour. A. W. W. A.*, **30**: 1557). Clarified water issues vertically from the sludge blanket and flows to the top of the basin for removal by weirs. Sludge is held in suspension by agitation and velocity of flow, throughout the mixing and clarification zones, thus permitting free contact of the treated water with the surface of the sludge particles. The basins are usually circular in plan, although the equipment has been successfully adapted to rectangular basins. The inner cones in Illinois have been constructed of steel plate and the basin proper of reinforced concrete. In small installations the entire structure may be fabricated from steel and in large units reinforced concrete may be used throughout as at the Minneapolis plant now under construction.

### Similarities of Equipment

It is of interest to notice the similarity of the Precipitator and the Accelerator. While mechanical details differ, both types employ a slurry of previously precipitated sludge particles, with which the treated water is intimately contacted, while clarification is effected more by mechanical means than by sedimentation.

The design of both the Precipitator and Accelerator has been based upon water surface area over the clarification compartment, rather

than upon volumetric capacity. The design rate has varied from slightly less than 1 gallon per square foot per minute in the earliest installations, up to 2 gal. per sq.ft. per min. in some recent designs in Illinois. Currently, the basic design rate of the Precipitator is about 1.25 gal. per sq.ft. per min. and Accelerators from 2 to 2.5 gal. per sq.ft. per min. It appears to the writers that the critical velocity occurs in both types at the top of the sludge blanket and not at the water surface. Therefore, it is probably improper to compare these two types of treatment upon the above numerical values, because the basins are of different shape. The Accelerator has vertical basin walls and the sludge blanket area is approximately the same as the water surface. The Precipitator has sloping walls on both inner and outer cones, resulting in a much greater rate in gallons per square foot at the sludge blanket than on the water surface.

### Design Rates

The area method of rating is similar to that used for sand filters and it is interesting to notice that the rates are approaching and even exceeding that generally accepted for rapid sand filtration, thus decreasing the space requirement for treatment works. As data is obtained from the plants in operation, there is a tendency to increase the design rate of the newer installations. Under favorable conditions one plant in Illinois has been operated at rates up to  $3\frac{1}{4}$  gal. per min. per sq.ft. of surface area without raising the effluent turbidity above 3.0 parts per million. However, the water temperature was high (82°F.) with resultant low water viscosity. When the critical velocity through the sludge is approached it is apparent that water temperature, through viscosity, may affect maximum rates the same as in sand filter washing.

The choice of a design rate should be based upon a study of several factors, including the water temperature, raw water turbidities, and the raw water mineral character, especially the magnesium content. When large amounts of magnesium are precipitated as magnesium hydrate, efficient clarification may be expected at lower rates than when calcium carbonate alone is removed. It is the opinion of the writers in view of present information that design rates should be chosen conservatively and rates above those now prevailing, adopted with caution. Under favorable conditions of water character and temperature and with exceptionally good operating personnel, higher design rates may be justified, but for average municipal instal-

lations a liberal factor of safety should be maintained at least until such caution is definitely shown by experience to be unnecessary.

There are several factors which control the rate at which a unit may be operated. The maximum velocity at which water may issue from the top of the sludge blanket in the clarification compartment, without excessive turbidity, apparently depends upon the character and quantity of sludge and to some extent upon water viscosity, which varies with water temperature.

### Size of Sludge Particles

Sludge particles in a properly developed slurry are considerably larger than obtained with ordinary mixing and sedimentation and the particles appear to be more dense. The sludge often feels "gritty" when rubbed between the fingers or against a glass surface. The larger particle size substantiates the contention that calcium carbonate is deposited upon the existing sludge particles by crystallization, tending to produce a more nearly stable basin effluent. A sample of sludge collected by the writers from one of these plants in Illinois after several months' operation, had the appearance of very small filter sand. The grains were dense, well rounded, almost perfect spheres, and practically pure calcium carbonate (see fig. 2). However, slurry composed of these exceptionally large particles is not desirable. If allowed to grow indefinitely the agitation equipment may become overtaxed and the slurry is not as effective. In any slurry of given sludge concentration there is less particle surface exposed to the water when the particles are too large. If the particles are very small the maximum in particle surface exposure is obtained, but clarification efficiency decreases. The optimum is a happy medium between these two extremes and must be determined by operating conditions. Sludge character and particle size may be controlled by proper agitation, chemical dosage and sludge removal, although raw water character must necessarily dictate certain elements of sludge character and chemical dosage. A water high in magnesium must be overtreated with lime which produces a larger portion of magnesium hydrate particles, that are less dense than calcium carbonate particles, thus changing the general character of the slurry. Raw surface water containing turbidity and requiring larger amounts of coagulant may likewise alter sludge character.

The use of a coagulant has generally been advantageous at short-period Illinois plants and alum is usually chosen, although ferrous

sulfate has been used successfully on certain waters. Waters low in magnesium sometimes soften so readily with lime that the required hardness reduction is obtained without producing a magnesium hydrate floc, yet the pH may be too high for good alum floc formation. Use of one of the iron salts is generally successful under such conditions. One short-period plant in Illinois normally operates without coagulant, although alum was used for the first few days opera-

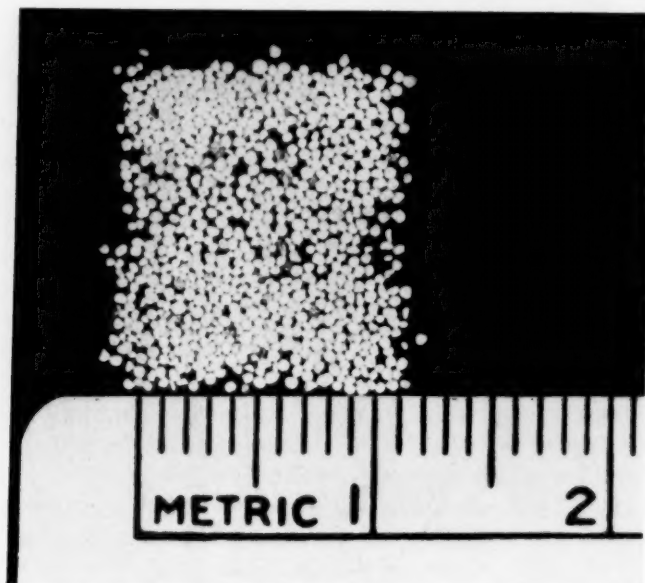


FIG. 2. Enlarged Sludge Particles. NOTE: Particle size here indicated is greatly in excess of the desired optimum. Illustration shows crystal growth does occur and the extent to which it may progress if not controlled. Smallest divisions on scale are 1 millimeter.

tion, until a suitable slurry was developed. The raw water, obtained from a well, contains 6 p.p.m. iron. This plant as operated removes very little magnesium. Where magnesium is high, alum seems to be of considerable assistance, just as in conventional lime softening plants. Whether this is because of improved coagulation or through catalytic action we have not determined.

Both Precipitator and Accelerator installations utilize continuous sludge withdrawal. The Accelerator incorporates an internal sludge

concentrator which is simply a submerged weir on one side of the clarification compartment with the crest at the sludge blanket level. As sludge accumulates and the slurry is expanded, it overflows into the concentrator compartment. As there is relative quiescence in the concentrator, solids concentrate at the bottom and flow to waste under head produced by water in the basin. Valve opening adjustment varies the discharge rate. Control is usually manual. In addition to continuous sludge removal in the Accelerator it is usually advisable to open the main sludge drain valve for a few seconds only each day. This tends to remove the very large particles which may have built up by crystallization to a size too great for the agitator to keep in suspension. Otherwise, oversize particles may accumulate at the bottom of the basin and if not removed may eventually stop the agitator. Without proper control there is a tendency for the coarse particles to displace the small ones in the slurry and as previously suggested a sludge composed of relatively large particles presents less crystal surface to the water being treated.

### Removal of Sludge

In the Precipitator, sludge is removed continuously from the bottom of the basin and particles too large to remain in suspension tend to be automatically discharged. Concentration of removed sludge when effected has been accomplished by an external concentrator. This is essentially a plain sedimentation basin in which sludge thickening occurs. Concentrated sludge is removed from the bottom for final disposal. Supernatant liquor from the top is pumped back to the treatment basin influent. One Precipitator installation in Illinois does not practice sludge concentration. Here two basins are operated in a split treatment combination and the continuous sludge discharge averages about 3 per cent of the total water treated. Where concentration is utilized at either type of plant, the loss through sludge discharge may be reduced to about 1 per cent or less. This item varies with the sludge character. Sludge containing large amounts of magnesium hydrate is more difficult to concentrate than one made up principally of calcium carbonate.

Percentage of solids in the slurry varies under different conditions from about  $\frac{1}{2}$  to 2 per cent by weight, although heavier concentrations have been used. Physical properties of the slurry vary with the type of water being treated, the chemicals used and the relative amounts of calcium carbonate and magnesium hydrate precipitated.

The proper slurry character, which must be maintained to produce optimum results, has in Illinois been determined generally by trial, the chemical dosage and slurry concentration being adjusted and results observed until the maximum efficiency is obtained.

Of the two operating installations of Precipitators in Illinois, both plants have more than one basin, so arranged to permit either series or parallel operation. Accelerator installations so far have been confined to single basin plants. While treatment in a single basin has produced reasonably satisfactory results, it appears to the writers that series operation of two basins may effect operating economy, when waters of certain characteristics are treated. As previously stated, waters high in magnesium require excess lime treatment in short-period plants, as in conventional lime-soda installations. Also, low temperature water may require considerable overtreatment with lime, while the same water at higher temperatures may sometimes be satisfactorily treated without obtaining caustic alkalinity in the basin effluent. Obviously the former would require recarbonation, while it might be eliminated for the latter. If overtreatment is necessary, series operation permits the use of the second basin as a "stabilizer." Secondary application of alum in this basin may greatly assist in reducing magnesium. Thus a surface supply incorporating multiple basins may utilize series operation in winter and parallel operation in summer, thereby obtaining increased plant capacity during high temperatures, when demand is greatest, yet operating throughout the year without recarbonation. This is the method Charles Spaulding has developed for operation of the Springfield, Illinois, plant.

### Split Treatment

If considerable overtreatment with lime is necessary, as for high magnesium waters, a second basin permits the use of split treatment, part of the untreated raw water being applied to the overtreated effluent of the primary basin; the secondary basin effecting final clarification and stabilization. Stabilizing action thus produced is similar to that accomplished in the ordinary marble test of the laboratory. Split treatment may be especially valuable in softening sodium bi-carbonate waters. Whether the second basin is used with split treatment or in straight series operation with secondary application of alum, it provides a means of producing a practically stable effluent before filtering, without recarbonation, even though excess lime

treatment has been employed. A cost study for any proposed installation should be made to determine whether the additional cost of a multiple basin installation is justified to obtain increased operating economy.

### **Aeration for Methane Gas Removal**

One upward flow plant in Illinois experienced considerable difficulty in the early months of operation from dissolved methane gas in the water. The raw water contained about 12 cubic feet of methane gas per 1,000 gallons. Under atmospheric pressure, this represents considerable super-saturation and the water effervesced like ginger ale. The plant was originally designed without aeration. The methane was released in the basin and in addition to the explosion hazard, small bubbles of gas rose vertically through the water, carrying particles of sludge to the surface. Satisfactory clarification was impossible, until aeration was installed to evolve the methane from the water before softening. Such a condition should not be encountered often.

Incidentally, aeration on Illinois ground water is usually justified before lime treatment, because the free carbon dioxide is generally quite high; the saving in lime dosage usually exceeding increased pumping and investment cost. Coke trays have generally been used and complaints from corrosion through excess dissolved oxygen from this cause have been few, although red water troubles have been experienced at some softening plants, usually where aeration was effected by some means other than coke trays, or where a surface water naturally contained dissolved oxygen to saturation.

In our experience these plants are more exacting in their requirements for accurate control of chemical dosage than the conventional lime plants. Variation in pumping rates should be accompanied by corresponding change in chemical feed. Uncontrolled fluctuation of either pumping rate or chemical application is almost immediately apparent in the basin effluent. Likewise when conditions are corrected the effects are rapidly reflected in the results produced because of the short retention period. Chemical feed equipment should be chosen which will give the least possible variation from any specific setting, and provisions should usually be made for maintaining a constant raw water rate, automatic controllers being installed for this purpose, if necessary.

All installations of both Accelerators and Precipitators in Illinois,

## ACCELERATOR

MUNICIPALITY	SOURCE	PLANT DATA (AS OPERATED)								
		Capacity (g.p.m.)	Hours Operated Daily (Approx.)	Aeration	Area Rate at Water Surface (gal./sq.ft./min.)	Retention Time		Chemicals (p.p.m.)		
						Total Mix. and Clar. (minutes)	Recarbonation (minutes)	Lime (H = Hydrated)	Soda Ash	Coag. (A = Alum; Fe = Ferric; Cop = Copperas)
Newman <sup>1</sup>	Well	75	6	Coke trays	0.76	111	12	495-H	None	17-A
Okawville <sup>2</sup>	Well	35	6	None	0.58	157	None	280-H	86	None
Chandlerville <sup>2</sup>	Well	50	4	None	0.83	110	9	467-H	240	12-A
Anna <sup>4</sup>	Wells	410	10	Coke trays	0.78	147	18	260-H	None	12-A
Marco <sup>5</sup>	Well	90	6	Coke trays	2.08	42	2.5	480-H	None	8-A

Notes on Accelerator Installations:

<sup>1</sup> Raw water contains 236 p.p.m. sodium bicarbonate, also 12 cu. ft. per 1,000 gal. of methane. Results from State Health Dept. analyses.

<sup>2</sup> Results from State Health Dept. analyses. No recarbonation facilities at this plant.

## PRECIPITATOR

MUNICIPALITY	SOURCE	PLANT DATA (AS OPERATED)											
		Capacity (g.p.m.)	Hours Operated Daily (Approx.)	Aeration	Area Rate at Water Surface (gal./sq.ft./min.)		Retention Time				Chemicals (p.p.m.)		
					Primary	Secondary	Total Mix. and Clar. (minutes)	Primary Mix. and Clar. (minutes)	Secondary Mix. and Clar. (minutes)	Recarbonated (minutes)	Lime (H = Hydrated; Q = Quick Lime)	Soda Ash	Coag. (A = Alum; Fe = Copperas)
Woodstock <sup>6</sup>	Wells	1000	12	Aer-O-Mix not used	1.05	1.31	169	94	75	None	490-H	None	9.5-A
Springfield <sup>7</sup>	Lake	4180-5050	24	None	1.19-1.44	None	84-102	84-102	None	None	84-Q	None	6.9-A
Springfield <sup>8</sup>	Lake	4180-8375	24	None	1.19-2.38	None	51-102	51-102	None	None	87.5-Q	None	12-A
Springfield <sup>9</sup>	Lake	4180-5050	24	None	1.19-1.44	None	94-102	84-102	None	29-36	120-Q	None	28.4Fe
Springfield <sup>10</sup>	Lake	4180-8375	24	None	1.19-2.38	1.19-2.38	102-204	51-102	51-102	None	102-Q	None	12.7-A

Notes on Precipitator Installations:

<sup>6</sup> Plant has 2 treatment units operated in series using split treatment. 80 per cent raw water to 1st basin, 20 per cent by-passed to 2nd basin; chemicals applied to 1st basin. Recarbonation equipment not used. 2 p.p.m. sodium hexametaphosphate applied to filter influent. Results are daily plant averages for April, 1939. Magnesium by State Health Dept.

<sup>7</sup> Summer operation with water temperature high. Results are daily plant averages for July, 1938. Basins operated in parallel. Rates given show max. and min. per basin.

# RESULTS

MUNICIPALITY	OPERATION AND ANALYTICAL DATA																	
	Iron as Fe (p.p.m.)		Water Temp. (°F.)		Turbidity Clarified Eff. (p.p.m.)		Alkalinity as CaCO <sub>3</sub> (p.p.m.)				Hardness (p.p.m.)				Magnesium as Mg (p.p.m.)		pH	
	Raw		Raw		Raw		Raw		Clarified		Final		Total		Non-Carb.		Mg as CaCO <sub>3</sub>	
	Raw		Raw		Total		Total		Phen.		OH		Total		Phen.		Raw	
	Raw		Raw		Total		Total		Phen.		OH		Total		Phen.		Raw	
17-A	Newman	3	56	8	540	356	168	0	308	134	304	74	236	234	60	31	14.5	7.1
None	Okawville	6	54	5	336	90	14	0	86	12	469	141	133	55	100	29	24.3	7.2
12-A	Chandlerville	0.68	54	6	288	110	92	74	40	9	517	64	229	24	28	50	6.9	7.0
12-A	Anna	0.12	52	8	280	38	24	10	34	11	311	83	31	49	40	21	9.7	7.2
8-A	Maroa	2.8	55	7	396	99	77	55	45	20	440	77	44	32	44	45	10.8	7.1

\* Results from State Health Dept. analyses. Raw water high in magnesium (50 p.p.m.).

\* Results are average of daily plant analyses for April, 1939. Magnesium determined by State Health Dept.

\* Alkalinities are daily plant averages for April, 1939. Hardness and magnesium by State Dept. of Health.

# RESULTS

MUNICIPALITY	OPERATION AND ANALYTICAL DATA																	
	Iron as Fe (p.p.m.)		Water Temp. (°F.)		Turbidity (p.p.m.)		Alkalinity as CaCO <sub>3</sub> (p.p.m.)				Hardness (p.p.m.)				Magnesium as Mg (p.p.m.)		pH	
	Raw		Raw		Raw		Raw		Clarified Eff.		Applied		Total		Non-Carb.		Mg as CaCO <sub>3</sub>	
	Raw		Raw		Primary		Secondary		Total		Phen.		OH		Total		Phen.	
	Raw		Raw		Primary		Secondary		Total		Phen.		OH		Total		Phen.	
5-A	Woodstock	1.4	56	—	30	4	382	114	87	60	98	45	0	98	45	99	43	371
9-A	Springfield	0	79	24	3.3	—	117	36	18	0	—	—	—	36	18	32	15	165
2-A	Springfield	0	66	13	5.8	—	125	35	18	1	—	—	—	35	18	32	17	170
3.4Fe	Springfield	0	49	6	6.2	—	130	48	35	22	—	—	—	40	14	34	11	179
7-A	Springfield	0	48	12	9.1	1	129	75	45	15	39	20	1	39	20	33	17	177

\* Results are daily plant averages for October, 1938. One basin only used entire month, max. rate used about 5 hr. daily. No noticeable turbidity increase for these periods.

\* Results are daily plant averages for January, 1938. Shows winter operations with basins in parallel. Re-carbonation used.

\* Results are daily plant averages for January, 1939. Shows winter operation with 2 basins operated in series. Split treatment not employed. Alum applied only to 2nd basin influent.

now operating, treat ground water having a constant temperature and quality, except one plant using a large artificial lake, where changes of raw water character are relatively slow and sudden high turbidities, characteristic of mid-western streams, are not obtained. Thus the raw waters treated in these plants in Illinois have not been those that change character rapidly. How difficult it may be to control the treatment in a plant utilizing a surface source of "flashy" character, we do not know. One municipality in Illinois is now installing a single upward flow unit to effect softening, on a surface water supply, so arranged that the existing plain sedimentation basins may provide preliminary settling, if necessary. Here will be an opportunity to study the effect of high raw water turbidities (3,000-4,000 p.p.m.) in short-time upward-flow treatment. All short-period upward-flow plants in Illinois are used for softening purposes, one combining water purification with the softening. While several installations are under consideration for surface water clarification only, none have yet been installed in this state. Two new upward flow plants are now under construction to effect softening on surface waters. One other plant recently completed is not listed in the tabulation, because sufficient operating data is not yet available to be of value. (See tabulation on preceding pages.)

#### Technical vs. Non-Technical Operators

Most of the upward flow plants under our observation have had problems to solve when the installations were placed in operation, although this is not uncommon for a water treatment plant of any type. Adjustments have usually been necessary and in some of the earliest plants extensive mechanical changes were effected before results considered satisfactory, were obtained. Recent installations, of course, have profited by the experience of the early plants, both in design and operation. While only one short-time plant in Illinois is operated by technically trained personnel, reasonably satisfactory results have been obtained at all installations, following the usual preliminary period of adjustments. While maximum efficiency has probably not been attained at all plants operated by non-technically trained persons and the solution to all operating problems has not always been determined as quickly as might be desired, it must be recognized that these plants are relatively new and there are yet very few data available to guide operation procedure.

### Conclusions

May we suggest, in conclusion, that short-time upward-flow treatment appears to offer a method of solving some of our water treatment problems in an efficient and economical manner. While there is undoubtedly much to be learned about the process both in design and operation, there is now available sufficient operation experience and history of reasonably satisfactory results to warrant consideration of this method of treatment, when water softening is contemplated. In the opinion of the writers, the basin principle of short time upward flow softening gives fair promise toward the complete elimination of recarbonation, even in high magnesium waters requiring excess lime treatment.

The writers express sincere appreciation to the many persons whose valuable cooperation and assistance have made this paper possible.

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**Discussion by W. A. Kramer.\*** Mr. Klassen and Mr. Spafford have described and presented the results and their experiences with the Accelerator and Precipitator in an excellent manner. It is impossible to compare the efficiency of this type of plant with the more or less conventional type, unless the two types of plants were built side by side and were operated simultaneously on the same water. Such a procedure has been attempted by some but cylindrical tanks of relatively small capacity were used for the conventional plant. The setup for comparison could not be termed a conventional plant.

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To what extent the short-time upward-flow method of treatment is going to revolutionize water softening is uncertain. We are viewing the development from an experimental standpoint. The efficiency of this type of plant over the conventional type where primary and secondary treatment, two stage coagulation is practiced has not been proven. We are of these opinions because a study of the results of plant operation that have come to us does not reveal any exceptional efficiency.

### Evolution of Upward-Flow Method

It may be of some interest to review the evolution of this method of treatment as applied today. The idea as near as we can learn was first conceived by Charles Herschel Koyl about 1900. In his patents of this principle and certain design of tanks it is stated: "I have shown the value of allowing or compelling the water from the reaction tank of a water softening machine bearing fine, freshly formed precipitates to rise through a bed of old precipitate which acts as a strainer to retain and to hold the new fine precipitates. Further one of the beneficial results of carrying this expedient into practice is that I am able to use settling tanks of very much less size and capacity than would otherwise be possible."

Mr. Koyl referred to his tank as an agglomerating tank instead of by some of the more popular names we hear today.

Perhaps because of the growing demand on the part of the public for soft water, particularly in the smaller cities that seek softened water for low cost, there has been extended activity in the softening field. Engineers have sought to justify the cost and reduce initial expenditures in the construction of plants to meet scanty budgets during depressed economic conditions. So after a dormant period of approximately forty years this process is revived and represented in terms of considerable optimism. This process of softening and treatment was used by some of the railroads. Why it never met with more satisfaction and favor is not clear. It is evident however that the system will now become more common partly because of the expert promotional work being done and because of the desire of those interested to give the method a thorough testing.

There are two upward-flow treating plants in the State of Missouri at the present time. One for softening a well water and the other on clarification of an impounded reservoir supply.

The softening plant encountered numerous difficulties as regards

control of sludge blanket, maintaining an effluent of reasonable uniform character, and regulation of carbonation in the short period provided for this phase of the treatment. The plant in operation for clarification was designed for an impounded reservoir supply and is the first plant installed of this type purely for clarification. The results from the application of this principle to turbid water were therefore questionable and uncertain and it was felt that some guarantee should be made to the city that reasonable results would be obtained. The company made a guarantee of turbidity removal of 3,000 p.p.m. to 25 p.p.m. There were no limitations as to the chemical changes. This guarantee did not mean a great deal for if excess chemicals were applied it might be possible to settle a water of turbidity 3,000 to 25 in much less time than can be done in a conventional type plant with lower chemical dosages and there was no assurance that efficient coagulation would be obtained or that water of a turbidity of 25 p.p.m. would filter satisfactorily.

Operating data are too incomplete to justify definite conclusions. Since the plant has been installed the maximum turbidity reached in the raw water is approximately 1,000. With charges of approximately 8 g.p.g. of lime and 7 g.p.g. of alum the minimum turbidity passing to the filters was 30-35 p.p.m.

#### **Increase in Hardness**

During this period the hardness was increased from approximately 85 to 115 p.p.m. When the raw water turbidity is 50 p.p.m. and with approximately 2.5 g.p.g. of lime and 2.0 g.p.g. of coagulant it is reduced to 10 p.p.m. going to the filter with an increase in hardness of 68 to 85 p.p.m.

Clarification has not been attained without an accompanying increase in hardness. Experiments are being conducted to accomplish satisfactory clarification without an increase of hardness. Clarification and adequate settling have not been attained with less than 2 g.p.g. of coagulant. It appears this minimum is necessary to maintain the sludge blanket regardless of the low turbidity that would ordinarily require less coagulant in a conventional plant. A charge of 2 g.p.g. is comparatively high for the character of the water treated in this impounded supply, when the turbidity is 50 or less. Perhaps one of the chief difficulties in this plant from an operating standpoint is maintaining the sludge blanket. As already pointed out the effects of treatment in these plants are rapidly reflected through the plant

and operation is more exacting in control and attention. Where operated intermittently the problem of control and maintenance becomes more difficult and the applicability questionable.

The upward-flow treating plants make a neat setup from the standpoint of architectural and engineering design. The theory as applied to softening is good. Finances, conditions of water treated, operating personnel, and other local factors will govern the choice of the type of plant selected. No doubt there will be conditions that justify the upward-flow treating plants. However we believe their selection should be based on careful studies of the characteristics of the water to be treated as well as detailed engineering and economic comparison with other methods.

**Discussion by S. B. Applebaum.\*** We must compliment the authors of this paper for its clarity and the soundness of the engineering judgements contained therein.

We are in agreement with the authors as to the desirability of conservative design in selecting the rates of flow at the surface of the tanks to avoid sludge carry-over. For surface supplies that approach freezing temperature it is our practice to use a rate at the top surface of the tank of 1.25 gal. per min. per sq.ft. For warmer non-freezing well waters, or other preheated waters, higher rates may be employed. If the magnesium content of the sludge is abnormally high, the top surface rate should be decreased. Further study will be required to establish the optimum rates for all conditions of temperature and sludge composition. Until further experience is obtained from actual installations over a wide range of water composition it is desirable to be conservative in selecting these rates. Furthermore, it must be noted that the rate as measured at the top of the sludge blanket may be the same in various types of design but in the Spaulding design, due to the sloping sides above the top of the blanket, the velocity of flow still further decreases after the water leaves the top of the blanket so that when the water reaches the top surface of the tank to be drawn off, it has decreased in velocity appreciably. This provides an additional factor of safety to prevent sludge carryover.

We also agree with the authors in the desirability of using two-unit reaction tanks so that they may be operated either in parallel or in series. This is especially important where the raw water contains

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appreciable magnesium in which case the split lime treatment may profitably be employed, adding all the lime required to a portion of the water entering the first Precipitator, bypassing the remainder of the raw water around the first Precipitator. The free carbon dioxide and bicarbonates in the bypassed portion of the raw water would then act to neutralize the excess hydrates in the effluent of the first Precipitator. This split lime treatment using two stage series arrangement avoids the use of carbonation equipment.

The authors have intimated that the Spaulding design used only external sludge concentrators. This is true at the plant at Springfield, Illinois, where the common external sludge concentrator is employed for three Precipitators. However, there is nothing intrinsic in the Spaulding design which makes it necessary to use external sludge concentrators. We have installed Precipitators with internal sludge concentrators. The operation of the concentrator is the same whether it is inside the Precipitator or outside of it.

The authors also recommend that the rate of flow of water through the reaction tanks be maintained as constant as possible for simplicity of control of chemical feeding and maintenance of the sludge blanket. As regards chemical feeding, proportionating chemical feeds are available which take care of variable flow rates. As regards maintenance of sludge blanket, a constant flow rate is unnecessary with the Spaulding design of Precipitator because the sloping sides of the Precipitator insure the maintenance of a sufficient depth of sludge blanket with wide variations in the rate of flow. The horizontal cross-sectional area of the sludge blanket rapidly decreases as the apex or port at the bottom of the sludge blanket is reached. Therefore, even at extremely low rates of flow there is always some depth of sludge blanket present. This is one of the advantages of using sloping sides as recommended by Mr. Spaulding.

Mr. Kramer in his discussion of this paper stated that there was no evidence that this type of design saves any chemicals or improves chemical results. It is true that it is difficult to collect evidence of this kind due to the fact that in order to be convincing such evidence must be obtained with the older design and the newer design operating on the same water supply under the same conditions. This is usually difficult to obtain. However, we have been fortunate in obtaining two cases of this type which supply this kind of evidence.

The first case is the plant at Woodstock, Illinois, where the same well water supply was previously treated in an older system.

Then the same settling tanks were remodelled into the Spaulding design and the results have not only been improved, but a considerable saving of chemical resulted. Table 1 gives the comparison.

The second case is the Gulf States Utilities plant at Baton Rouge. Here the Mississippi River water has been treated with lime for about 10 years. Three steel tanks 34 ft. in diameter by 54 ft. high were available for this purpose. One of these tanks was remodelled into a double-deck Louisiana-Spaulding Precipitator, (The design adopted was evolved with the cooperation of the user and Stone & Webster Engineering Corp.), and 1,800 gal. per min. was passed through this tank; the two other tanks were left unchanged and about 600 gal. per min. were passed through each of them. Therefore the retention period in the Spaulding unit was only one-third as great as in each of the other two units. All the water was treated with the

TABLE 1  
*Comparison of Consumption of Chemicals, etc.*

	REQUIREMENTS WITH CLARIFIERS	REQUIREMENTS WITH PRECIPITATORS	SAVING WITH PRECIPITATORS
Lime .....	30 gr./gal.	26 gr./gal.	13 per cent
Alum .....	1½ gr./gal.	¾ gr./gal.	40 per cent
Sludge Blowoff .....	6 per cent	3 per cent	50 per cent
Filter Wash Water .....	3 per cent	0.5 per cent	83 per cent

same amount of chemical in a flash mixer tank ahead of the 3 settling tanks so that all conditions of temperature, raw water composition and dosage remained common to the two systems. The chemical results in the old and new systems are given in table 2.

This table is evidence of the benefit to be derived from the use of the sludge blanket and other features of the Spaulding design.

It may be mentioned further that the first paper on this type of equipment presented by Spaulding and Timanus in 1934 (Jour. A. W. W. A., 27: 1935) cited operating results of a 1½ m.g.d. Precipitator having 58 minutes detention, which were compared with those of an older softening plant having actual mixing time of 46 minutes and settling time of 8 hours. The raw water received all chemicals which were mixed before division of the flow between the two systems. This comparison revealed an improvement of 14 per cent in softening results obtained in the Precipitator in spite of

the fact that the detention time was one ninth of that in the older plant which was equipped with mixing chamber, clarifier and settling basin. The results cited were averages of daily tests covering sixty days of operation and the tests were continued for nearly a year with similar results.

Mr. Kramer also raises the question of this type of design requiring more attention than the older type. Our experience with a considerable number of Precipitators installed during the last year is that once the plant is tuned up and properly adjusted, the attention

TABLE 2  
*Comparison of Results*

	RAW	SPAULDING UNIT	OLDER UNITS EACH
Flow Rate, gal. per min. per unit...	—	1,800	600
Total Hardness as $\text{CaCO}_3$ .....	186 p.p.m.	92 p.p.m.	106 p.p.m.
Methyl Alkalinity as $\text{CaCO}_3$ .....	134 p.p.m.	34 p.p.m.	49 p.p.m.
Phenol Alkalinity as $\text{CaCO}_3$ .....	0 p.p.m.	20 p.p.m.	30 p.p.m.
Hydrate Alkalinity as $\text{CaCO}_3$ .....	0 p.p.m.	6 p.p.m.	11 p.p.m.
Turbidity .....	142 p.p.m.	13 p.p.m.	22 p.p.m.
Temperature .....	61 °F.	61 °F.	61 °F.
CaO fed .....	0	118 p.p.m.	118 p.p.m.
Ferrisul fed .....	0	10 p.p.m.	10 p.p.m.

is no greater than with the older type. This is borne out by the fact that we have installed a considerable number of Precipitators in small carbonated beverage plants where non-technical help is available for operating these plants. In general, the smaller a lime treatment plant is, the greater is the skill required to control it. Nevertheless, these bottling units have gone into operation very satisfactorily with surprisingly small amount of assistance on our part. We have also installed a considerable number of Precipitators in small towns and villages where the available operators are non-technical and similar satisfactory results have been obtained.

**Discussion by A. S. Behrman.\*** The only discussion which can properly be made of this paper is commendation for its unusual excellence both of subject matter and of presentation. Mr. Klassen and Mr. Spafford have been in a uniquely favorable position to make an

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intensive study of this most recent development in lime-soda water softening; and the scientific analysis and evaluation of this development, and the wealth of specific information given in the paper, provide ample evidence that they have used this opportunity to good advantage.

I believe it only proper to add that the same scientific approach to the subject displayed in this paper has also characterized the official attitude of the authors towards new projects involving the use of accelerated lime-soda softening. The Illinois State Board of Health was quick to recognize the technological and economic importance of this advance, not only to the water softening art but also to the communities affected. It is a pleasure for me to acknowledge on behalf of our own organization the scientific spirit which Mr. Klassen, Mr. Spafford and their associates have taken in connection with the numerous Accelerator installations in Illinois. They have been entirely fair and impartial at all times. They have been critical, when criticism was indicated. They have had to be shown, and shown very definitely, the fundamental soundness of what we were undertaking to do; but, once convinced, they have been very helpful and cooperative. This same attitude, I am glad to report, is being taken by more and more State Boards of Health throughout the country towards advances in the water purification art; and such an attitude not only reflects credit on a State Board of Health, but is also to the best interests of the state and communities which it serves.



## **Performance of Upward-Flow Basins At St. Petersburg**

*By Robert W. Sawyer*

**T**HE upward-flow lime-softening and coagulation-settling tanks of the Pinellas Water Company which serves St. Petersburg, Florida, have now been in operation for over four years. The softening plant has a rated capacity of twelve million gallons daily and contains the following steel tank units built on the ground: three lime softening tanks, two of them 50 ft. in diameter and one 75 ft. in diameter; two coagulation-settling tanks, 60 ft. and 91 ft. in diameter; two filters 50 ft. in diameter; one mixer 10 ft. in diameter; a storage bin for coagulant of a diameter of 10 ft.; a storage bin for lime, 15 ft. in diameter; and a laboratory and service building.

Cost of these items complete with piping, valves, embankment to form sludge basin, roads, walks, and all overhead costs amounted to \$20,000 per million gallons of capacity. A clear water reservoir in the form of a 50-foot diameter steel tank with dropped roof supporting an aerator and a booster pump which is also used for washing filters had been in operation on the site for five years prior to the completion of the first unit of the softening plant in September, 1935 and are not included in the cost figures just cited.

The softening tanks and coagulation tanks are similar in design and welded steel construction was used throughout. Each tank is about 20 ft. deep and has a full depth circular center well varying from 10 ft. in diameter for the 50-foot tanks to 15 ft. in diameter in the 91-foot tank. The influent pipes enter tangentially near the tops of the wells and discharge is from the bottoms of the wells through radial pipes on the tank floor. A valved drain laid under

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each tank is connected into the bottom of each center well. Slotted collector rings at the top of each unit receive the settled water and carry it into the effluent pipe from the tank.

Figure 1 shows the arrangement of the radial distributor pipes on the bottom of the 91 ft. diameter coagulation tank.

The raw water is obtained from a system of wells driven into the Tampa limestone and yielding a water with a methyl orange alkalinity of 188 parts per million, soap hardness of 184 p.p.m. and carbon dioxide of 15 p.p.m. Sixty per cent of the water is aerated and

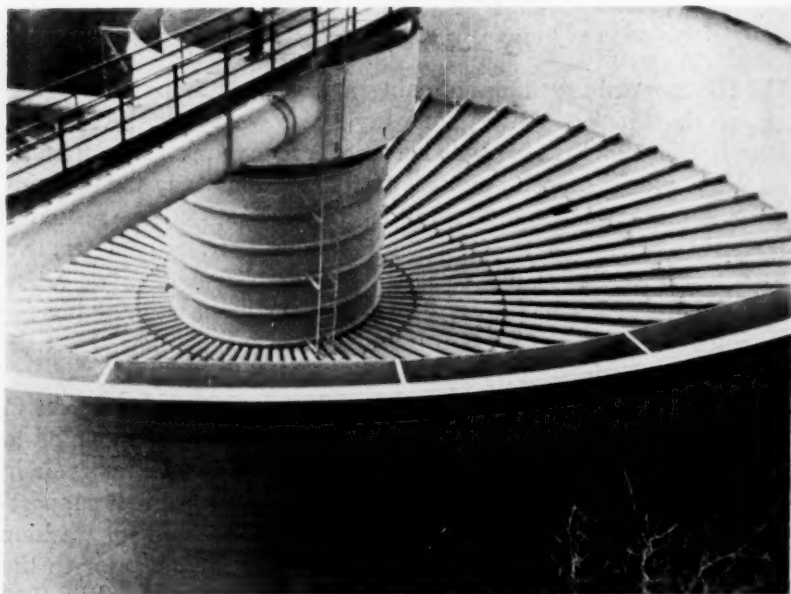


FIG. 1. Radial Distributor Pipes in Coagulation Tank

passed through the lime softening tanks, the remainder being by-passed to mix with the softened water to which the coagulant has been added.

#### Lime Softening Tanks

The aerated water containing 2 p.p.m. of carbon dioxide flows by gravity from the aerator flow through the influent pipes to the center wells where it receives its milk of lime dosage containing 600 to 625 pounds of quick lime per million gallons. Thorough mixing is

accomplished as the water swirls downward in the center well until it reaches the radial distributor pipes at the bottom. Even distribution over the tank area is accomplished by orifices spaced along the distributor pipes so as to deliver equal quantities of water to equal areas of the tank floor. The lime-treated water then rises through the suspended or floating filter comprised of previously precipitated calcium carbonate sludge in the bottom of the tank. The rate of vertical rise is one inch per minute when the plant is running at a 12 m.g.d. rate with 60 per cent or 7.2 m.g.d. passing through the softening tanks. The total rise is 16.5 feet from the distributor pipes to the slots in the effluent collector rings so that 3 hours and 20 minutes are required for the water to pass upward through the tank.

The softened water has 40 p.p.m. of hardness, pH of 9.8 and phenolphthalein alkalinity of 12 p.p.m.

Each tank is desludged about every three days by opening the drain valve and drawing the tank down 12 to 18 inches. During this process the distributor pipes in the bottom act as collectors for the sludge which enters the orifices at high velocity and is carried to the center well for discharge into the drain. Five or six times a year each lime tank is emptied for cleaning to remove the scale in the distributor pipes; the work requiring about 12 hours from the time the tank is taken out of service.

Loss of head in the lime tanks from the top of the center well to the effluent collector ring is three tenths of a foot which provides mixing in the well, distribution of the water through the radial pipes, loss through the suspended sludge bed and loss through the slots in the collector ring. In Fig. 2 the 75 ft. diameter lime tank is seen in the background with the two 50-foot lime tanks in the foreground. In the left center the aerator is seen in operation on the roof of the 50-foot filtered water storage tank.

### Split Treatment

The softened effluent from the lime tanks receives the coagulant dosage of "Ferrisul" (ferric sulfate) which is introduced against a head of 20 ft. of water just before the 40 per cent of pre-chlorinated, unaerated raw water is mixed with the softened water at the entrance to a 10-foot diameter mixer 20 ft. high in which thorough mixing of the two waters is obtained together with recarbonation by the carbon dioxide in the raw water and the beginning of floc formation.

### Coagulation and Settling Tanks

From the top of the mixer the coagulant-treated water is piped to center wells of the upward flow coagulation and settling tanks and mixing continues down through the center well and until the water passes out into the bottom of the tank through the orifices in the radial distributor pipes. At design capacity the rate of upward flow in each tank is  $1\frac{1}{2}$  in. per min. which will deliver an excellent settled water as demonstrated by operation of the 60-foot tank at this rate for a large part of the time over a three-year period. As with the lime tanks, desludging is accomplished by opening the drain



FIG. 2. Partial View of Plant Layout, Showing Three Lime Tanks and Aerator

valve long enough to draw 12 or 18 inches of water from the tank about every seven days. No other cleaning is necessary. Four pounds of activated carbon per million gallons are added to the water entering the coagulation and settling tanks all of which is removed in the contact sludge bed and appears to aid in stabilization of the sludge.

### Filters

Each of the 50-foot filters is divided into three beds by steel division walls which extend only to the wash troughs. Filters are washed at a loss of head of from 4 to  $4\frac{1}{2}$  ft., 6 to 7 m.g. being filtered

between washes. Clarity of the settled water is such that the gutters and sand are visible from the top of the filter through fifteen feet of water. There has been no indication of carbonate incrustation of the sand. The filtered water has a pH of 7.4; color, 5 p.p.m.; soap hardness, 95 to 100 p.p.m.; and methyl orange alkalinity of 85 to 90 p.p.m.

TABLE 1  
*Operating Data*

1938	M.G.D. MONTHLY AVGE.	M.G. MAX. DAY	M.G. MIN. DAY	QUICK- LIME BASED ON TOTAL WATER	FERRISUL FOR TOTAL WATER	RAW WATER HARD- NESS	FILT. HARD.	REDUC- TION
				lb./m.g.	lb./m.g.			
Jan.....	3.8	4.7	3.0	604	106	185	98	87
Feb.....	4.5	5.1	3.8	608	105	185	99	86
Mar.....	4.7	5.1	3.9	630	107	185	100	85
Apr.....	4.2	4.8	3.2	633	102	183	98	85
May.....	3.9	5.0	1.8	620	106	182	97	85
June.....	2.5	3.4	1.9	597	104	185	98	87
July.....	2.2	2.8	1.7	576	104	185	98	87
Aug.....	3.0	4.2	1.9	595	107	183	100	83
Sept.....	2.9	3.9	2.2	600	109	183	98	85
Oct.....	2.7	3.5	2.3	598	104	181	96	85
Nov.....	3.4	4.0	2.8	608	108	182	96	86
Dec.....	3.9	4.5	3.3	613	107	182	95	87
1939								
Jan.....	4.4	5.0	3.4	623	109	182	93	89
Feb.....	4.7	5.6	3.9	625	116	182	90	92
Mar.....	5.3	5.9	4.6	636	122	184	94	90
Apr.....	4.2	5.1	3.2	642	111	186	95	91
Average.....				613	108			84

### Cost

Based on the net water delivered from the plant to the city the average use of quicklime is 613 lb. per m.g.; "Ferrisul," 108 lb. per m.g.; activated carbon, 3 lb. per m.g.; and pre-chlorination, 7 lb. per m.g. The total cost of chemicals is \$6.86 per m.g. or 8.2 cents per million gallons per part per million of hardness removed. Other

costs which did not exist before the softening plant was installed include power for plant use and to overcome the total loss of head of 3 ft. plus loss through the filters, extra plant labor and miscellaneous small expense such as laboratory, fuel oil for heating water, etc. At present rates of output these fixed costs amount to \$2.35 per m.g. and with the cost of chemicals bring the total to \$9.21 per m.g. which is well below the preliminary design estimate made in June, 1934.



FIG. 3. Chemical Storage Bins and Service Building

Maintenance costs are extremely low and will probably contain painting as the principal item as time goes on, but this cost is slight. The original storage tank at Washington Terrace Pumping Station has not been repainted since it was built in 1930 and there is every indication that the paint on the softening units will give as good service.

Quicklime and ferric sulfate are stored in steel standpipes with delivery of the chemicals to storage by bucket elevators (see fig. 3). Each bin has a conical bottom at sufficient elevation above the floor

to give space for weighing hoppers and the feeding machines below them. To insure that weather-proof storage for quicklime and ferric sulfate had been provided in the steel structures each bin was tested and made tight under an air pressure of 3 lb. per sq. in. Shortly after the bins had been filled for the first time in 1935 a further test was applied by a tropical storm. This test confirmed the conclusions reached as a result of our home-made air-pressure test.

Milk of lime from the Omega feeder and slaker flows by gravity in a steel trough to steel pots set in ground near the lime tanks where dilution water is added. Delivery from the dilution pots to the tops of the center wells in the lime tank is achieved by sand eductors.

"Ferrisul" is fed by an International feeder into a welded stainless steel dissolving pot and thence delivered under pressure into the softened effluent from the lime tanks by a Pyrex glass eductor.

Figure 3 shows quicklime being unloaded into the bucket elevator for the lime storage bin on the left. The smaller cylindrical steel structure contains Ferrisul storage and feeding equipment and at the right is the laboratory and service building.

Operations are under the direction of Raymond G. Ridgely with the same plant operators, A. O. Burleson and Brooks Baldwin, who have run the supply works since they were built in 1930. Extra labor added with the start of softening and filtration is taken care of by two men with occasional extra help on days when lime tanks are cleaned out.

From our experiences with this plant, the boiler feedwater treatment plant for the West Virginia Pulp & Paper Company at Charleston, S. C., the Rockefeller Estate at Pocantico Hills, N. Y., and several experimental pilot plants we are convinced that the upward-flow design for chemical reaction and settling offers more economies and efficiency than can be obtained with horizontal flow basins or tanks. The intimate contact between newly treated water and previously precipitated sludge permits optimum results with minimum chemical dosages and furnishes a contact filter which removes colloidal and very fine suspended floc very efficiently so long as the rate of upward flow is not great enough to carry the sludge to the surface. In operation the top of the sludge bed assumes a horizontal plane several feet above the bottom of the tank and removal of precipitated material is substantially completed by the time the upward rising water leaves the top of this suspended contact filter. The process is by no means confined to circular tanks. In many

cases it can be adapted to increase the capacity or efficiency of existing units and in large plants designs based on upward flow will show material savings over designs of horizontal flow basins based on "flowing-through time" or displacement.

The upward-flow design also can be utilized to provide for desludging without emptying a basin or tank so that regular sludge removal from a unit requires about one minute for the operation of an hydraulic valve.

The idea of using upward flow methods and of passing newly dosed water through previously precipitated sludge for treatment and clarification of water and sewage is not new. In 1891 a British patent was applied for by William Lawrence and his application was approved in 1892. Mr. Lawrence's claims appear to the writer to embody the fundamentals of upward-flow treatment as we know it today.

Turbidity and color removal, softening, iron removal and certain types of waste disposal which involve coagulation and concentration of the precipitated sludge are some of the more common types of treatment which adapt themselves to efficient handling by the upward-flow method and it seems likely that this process will find increasing usefulness.



## **Standard Classification of Accounts And Standard List of Retirement Units**

*By Louis D. Blum*

**S**TANDARDIZATION of accounting has made considerable progress during the past five years. Not alone has such standardization received attention in accounting for utilities, but with the advent of the "New Deal" and the various commissions established thereunder, uniformity of accounting and reporting to governmental agencies has also advanced. Uniform systems of accounts for utilities are not new, but the uniformity suggested for presenting accounting data by the Securities and Exchange Commission for issuers of securities in various businesses is an innovation. From the viewpoint of the Commission, it is advantageous to have the arrangement of data and the reference numbers of schedules uniform, regardless of the nature of the business, in order that the location of information may be facilitated. In view of the fact that many reports are public records, any one interested in specific items and familiar with the uniform arrangement may locate such information without examining an entire report, which procedure would be necessary if the uniform arrangement of reports was not prescribed.

Uniform systems of accounts for public utilities, prescribed by the state and federal regulatory agencies have been revised from time to time during recent years to provide for changes in accounting and rate making theories. The National Association of Railway and Utilities Commissioners has also revised its uniform systems of accounts for electric, gas and water utilities within the past three years and issued uniform lists of retirement units for such utilities. The uniform system of accounts and list of retirement units recommended by the

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National Association of Railway and Utilities Commissioners are designed so that they may be adopted by State Regulatory Commissions with such changes as the Commissions deem proper under existing local conditions.

The purpose of this paper is to present a comparison of some of the features of the current N.A.R.U.C. uniform system of accounts with the classification adopted by that organization in 1922, and to comment on the list of retirement units.

### **Classes of Utilities**

The 1922 "Uniform Classification of Accounts" provided for a classification of water utilities into four classes, as follows:

Class A—Average annual operating revenues exceeding \$250,000.

Class B—Average annual operating revenues exceeding \$50,000 but not over \$250,000.

Class C—Average annual operating revenues exceeding \$10,000 but not over \$50,000.

Class D—Average annual operating revenues of \$10,000 or less.

The uniform system of accounts adopted in November, 1938, classified water utilities into two classes; class A includes those with annual operating revenues of \$250,000 or more and class B includes those with annual operating revenues of more than \$100,000 but less than \$250,000. Water utilities with annual operating revenues below \$100,000 are not classified in the N.A.R.U.C. system of accounts. The accounts which should be used by either class are designated by the letters A or B appearing before the account number.

### **Utility Plant Accounts**

Under the 1922 classification of accounts it was prescribed that "Fixed Capital" account shall include the cost of property; such classification did not forbid the inclusion, in an account designated as "Fixed Capital Not Classified by Prescribed Accounts," of undisturbed book values which had been carried prior to the effective date of a prescribed uniform system of accounts, and which may or may not have represented the actual cost of the property in service, nor did such treatment operate as a prejudice to any future determination of actual cost of the property.

The 1938 "Uniform System of Accounts" prescribes accounts for "Utility Plant" and "Utility Plant Adjustments."

The "Utility Plant" account is divided into the following subdivisions:

- Utility Plant in Service
- Utility Plant leased to Others
- Construction Work in Progress
- Utility Plant Held for Future Use
- Utility Plant Acquisition Adjustments
- Utility Plant in Process of Reclassification

"Utility Plant in Service" shall include the original cost of the plant owned and used by the Utility in its operations. The uniform system of accounts defines "original cost" as the cost of property to the person first devoting it to public service. Accordingly, the amount to be charged to this account, in the case of the purchase of a complete plant for cash, might represent a value entirely different than what was paid for the property. If property was the subject of several sales since it was first devoted to public service, the profits or losses arising from such sales would have to be eliminated in order to arrive at the value at which the plant should be stated in the "Utility Plant in Service" account. This account should be charged with the cost of additions which shall include materials, supplies, labor, services and overhead construction costs. When property is retired, utility plant should be credited and depreciation reserve charged, if the property is of a depreciable class and the cause of the retirement is one for which provision has been made in the depreciation reserve. Losses arising from the retirement of property for causes not factors in computing depreciation should, when authorized or directed by a Commission, be credited to "Utility Plant" and charged to "Extraordinary Property Losses." The charges in the latter named account should be carried as a "Deferred Debit" in the balance sheet and are subject to amortization by charges to operations upon authorization of the Commission.

The accounts "Utility Plant in Service," "Utility Plant Leased to Others," "Construction Work in Progress" and "Utility Plant Held for Future Use" are, as stated in the instructions to the "Uniform System of Accounts," designed to show the original cost of property falling under such classifications.

Upon the acquisition of utility plant as operating units or systems by purchase, merger, liquidation or otherwise, the difference between the cost to the acquiring utility and the original cost as herein-

before defined should be charged or credited to "Utility Plant Acquisition Adjustments."

The account designated as "Utility Plant in Process of Reclassification" is designed for use as a control account for utility plant at the effective date of the system of accounts pending transfer or distribution to appropriate other prescribed accounts. In other words, if an analysis of the plant is not available at the effective date of the system, the entire plant should be entered in this account and appropriate distribution made as soon as practicable.

Two accounts are provided to record, by either debit or credit, the amount by which the book cost of utility plant differs from the cost of the plant to the utility when the difference is not properly includable in other accounts. The "Utility Plant Acquisition Adjustments" account referred to above is designed for use in connection with acquisitions subsequent to the effective date of the system, whereas the "Utility Plant Adjustments" account is designed for use in recording information at the effective date of the system.

### **Retirement and Depreciation of Property**

Until recent years most water utilities did not provide for depreciation in their accounts but, in lieu thereof, provided only for actual retirements by means of periodic charges to income and corresponding credits to "Retirement Reserve." This method was intended to equalize from year to year, as nearly as practicable, losses incident to important retirement of buildings, dams, etc., or of large sections of identifiable units of plant and equipment. The 1922 "Uniform Classification of Accounts" did not require consideration of the cost of replacing minor parts in estimating the provision for retirements; such replacements, as distinguished from the cost of replacements of large units were treated as repairs or maintenance. The charges to income for retirements were to be based on the utility's experience and best sources of information and such charges were to be in sufficient amount to provide, during a period of years, a reserve against which losses sustained upon the retirement of property could be written off. As between utilities there was a wide variation in the amounts of retirement expense set up. Some of the bases for the determination of retirement expense were (1) a percentage of gross income less maintenance, (2) a fixed or varying amount which the utility considered adequate, and (3) a percentage of the value of the property. Many arguments have been advanced in favor of and

against the retirement method. A discussion of these arguments could not be presented without making this paper very lengthy.

In the 1938 "Uniform System of Accounts" the "Retirement Expense" and "Retirement Reserve" accounts have been eliminated and replaced by accounts for "Depreciation" and "Reserve for Depreciation of Utility Plant." The retirement method contemplates the establishment of a reserve, sufficient in amount to provide, during a period of years, for losses sustained when property is taken out of service. The depreciation method contemplates periodic charges to income and corresponding credits to "Reserve for Depreciation" in amounts representing the loss in value not restored by current maintenance. Depreciation, as defined in the "Uniform System of Accounts," includes the consumption or prospective retirement of utility plant in the course of service from causes which are known to be in current operation and against which the utility is not protected by insurance. The definition is further amplified by enumerating causes to be given consideration in determining depreciation, such as wear and tear, decay, action of the elements, inadequacy, obsolescence, changes in the art, changes in demand and requirements of public authorities. The uniform system does not prescribe the method for determining depreciation such as straight line, sinking fund, etc., or the rates to be used, but leaves such directions to the Commissions adopting the system of accounts.

At the time of retirement of depreciable property, the reserve should be charged with the book cost of property retired and the removal cost, and credited with the salvage value and other amounts recovered, such as insurance.

Insofar as the depreciation theory is concerned, much has been said pro and con. Some of the more recent classifications of accounts adopted by Regulatory Commissions indicate a definite trend towards depreciation accounting.

### Retirement Units

One of the most confusing phases of utility accounting is the question of whether a retirement of property should be charged to the depreciation reserve or to maintenance.

For the purpose of promoting greater uniformity in water utility plant accounting, the N.A.R.U.C. adopted a "List of Retirement Units" during the year 1938, to be used as a guide in distinguishing between items replaced through the plant accounts and those re-

placed through maintenance. The list of units was intended to represent those of maximum size, but was not intended to list all items of property which might be classified as retirement units. This was done purposely, so that the list could be expanded in cases where it was considered advisable.

One of the items which received careful consideration by the Committee on Statistics and Accounts of the N.A.R.U.C. was the unit of retirement for mains. There were varying opinions among the members of that Committee and the Committee on Classification of Accounts of the American Water Works Association, which Committee was represented by the writer at several joint meetings of both organizations. In the original draft of the "List of Retirement Units" only retirements of mains in excess of 50 feet were considered as units. The representatives of the American Water Works Association recommended that the retirement unit should be one length or more of pipe. For the purposes of the list it was finally agreed that the unit for retirement purposes of transmission and distribution mains be fixed at two or more standard lengths, including fittings. In view of the fact that it was not intended to formulate hard and fast rules, the minimum retirement unit for mains could be reduced to one standard length as conditions or circumstances warrant.

### Conclusion

Both the "Uniform System of Accounts" and the "List of Retirement Units" as adopted by the N.A.R.U.C. represent a substantial contribution toward obtaining uniformity in accounting for water utilities. These documents, no doubt, contain some provisions which would be difficult to apply under certain conditions, but as more experience is gained under actual working conditions, revisions can be made as they are found to be necessary.



## Consumers Accounting and Collecting

*By M. F. Hoffman*

ONLY in recent years has the procedure for accounting and collecting for water consumers been given constructive attention. The fact that large economies, as well as improved public relations, may be effected in the technique of meter reading, billing and accounting for consumers, together with sound business practice in collecting bills, is sufficient inducement for the betterment of present methods.

Water works executives have been quick to grasp the advantages offered by new means of pumping, purification and distribution. After all, these bear the major costs of operation. With the improvement of these technical functions at their peak, it is high time that study be given to the possibilities of a simple, economical and accurate plan for consumers' accounting.

The history of public utility accounting reads like a romance. Back in 1878, with the advent of the first commercial telephone, subscribers were billed regularly, at monthly or quarterly intervals. The bills were prepared by hand and the charges posted to a ledger, which was cumulative for years. As payments were made, these remittances would be posted as credits, and before bills were rendered for the ensuing period, balances were abstracted from the ledger and carried forward as delinquent accounts. This was very crude, but it was the best means known at the time. There was practically no control of accounts receivable, with the result that it was difficult to balance consumers' accounts at any time.

Manufacturers of office machines, working hand in hand with public utility management, have proved their ability to cope with the situation. With the improving efficiency of machines, which made

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them applicable to billing and accounting for consumers' revenues, notable changes were made. With the advent of the Addressograph, hand writing of the bills was eliminated, as the names and addresses of the customers were printed. Primarily, the register plan, together with the Addressographed bill, both completed by machines, with automatic proof of the billing, was set up. Payments were posted, and at the close of each billing period balances were abstracted to be added to the ensuing month's bills. The register sheet was prepared by listing the folio number of the consumers' accounts. The charges were posted simultaneously with those to the bills but no control was set up. These were usually taken off at the close of the billing period. Balancing was unusually difficult for an error in posting made it necessary to check all charges and credits for that billing period to locate the difference.

With the increase in the number of users of public utility services, among which were water users, it was necessary to simplify this work. Two plans most universally used as a result of this increase are the dual ledger card and bill plan, and the accounting stub plan. The use of the ledger card in conjunction with the bill is well known, and in this paper we will treat of the stub plan only, as its application may be used both with the ledger card-bill plan or with the register sheet-bill method. It is not to be the objective of this paper to illustrate the advantage of the stub plan over the register sheet or ledger card plan, as we are more or less familiar with the old method. It will merely be our effort to depict a simple, practical plan which will be less costly of operation, will give an absolute control of revenues, which will, through its flexibility, permit of a splendid relationship with the public, and which will allow the payment of water bills in installments, as well as the expeditious follow-up for prompt payment of bills. One of the chief results obtained from this plan, where the billing is of large proportion, is the prompt application of collection enforcement to delinquent accounts, so that these delinquent accounts do not become a serious problem for the water works executive.

#### **Stub Plan as Used by Cincinnati**

In order to present the picture for discussion, the method in use in the Cincinnati Water Works is described step by step.

There are 102,000 consumers' accounts, all metered. Of this number, 2,000 represent commercial users. The meters on these

accounts are read and billed monthly, as they are productive of 50 per cent of all revenue. This causes a uniform flow of receipts, which is ideal for budget purposes. Of the remaining 100,000 accounts, a little over an average of 11,000 bills are rendered every ten days. On these accounts the meter reading sheets are taken into the field from three to two weeks before the bills are to be rendered. The Addressographed bills, printed, dated and addressed on an Automatic Addressograph, are prepared before the meter sheets are taken out. They are checked so that there will be a bill for every sheet, or a meter sheet for every bill. The bill is comprised of an accounting record, bill with cashier's stub, and delinquent notice on postcard stock. After being printed and addressed the delinquent notices are cut off and retained until needed. The accounting record, representing a loose leaf ledger record, is completed at the same time that the charges are posted to the bill.

Upon the return of the meter sheet to the office, the consumption is checked and the charge computed. At this time, "stuck" meters are reported to the Meter Shop for investigation, and notices are sent to the consumers where excessive consumption is shown. Unusually excessive consumption accounts are checked up in three days after the notices have been mailed to consumers or owners, to see if proper repairs have been made. "Re-reads" are also prepared where it was not possible to obtain readings of the meters. Billing is started on the particular unit which has been read, so as to be completed two days before the bills are to be mailed.

Bills are rendered every ten days, approximately on the 4th, 14th and 24th, on a quarterly basis. This means that 11,000 bills are sent out every ten days, and each of these units is one-ninth of the total number of accounts. Meters are read quarterly on all these accounts, with the exception of approximately 20,000 accounts located in the basin of the city, which is the oldest part of Cincinnati, where the meters are read monthly, although bills are rendered quarterly. This is for the protection of non-resident owners of large tenements, or small two-or-more-family houses. The bills are due on the 14th, 24th and 4 of the month, respectively.

Our experience indicates that approximately 80 per cent of these accounts are paid before the due dates. A delinquent notice, which is a byproduct of the original bill, is mailed, without any amount being indicated, in a pennysaver envelope, approximately three days after the expiration of the due date. These notices state that the

bills have not been paid, and that they must be paid by a date seven days after the mailing of these delinquent notices or an additional charge will be made for collection service. At the expiration of the final due date, the unpaid accounts are given active treatment. Delinquent bills are prepared on typewriter, and there is a charge of \$1.00 in the city, and \$1.50 in the county, set up for collection service. These bills are then given to turnkeys for collection. The turnkeys are bonded, and they endeavor to cover the entire district in one day. Of approximately 300 bills for which shut-off service has been arranged, at the end of five days all have been paid, or arrangements made for payment, with but three or four shut-offs being required in order to close the accounts.

As bills are prepared on the Burroughs Billing Machines, the charges are automatically set up. Journal entries are prepared for the general bookkeeper on the day the bills are mailed, showing the amount of the billing. The bills are all mailed on the evening of the 4th, 14th and 24th, and are in the hands of the consumers on the mornings of the 5th, 15th and 25th. By mailing these bills under a permit, we have return service from the Post Office in case a bill is non-deliverable. These services are ordered off; thus it is soon known whether the premises have been vacated, or a new application should be obtained. This cycle billing arrangement is ideal, in that it eliminates a peak load at any time and permits of a minimum personnel handling a maximum number of accounts.

#### **70 Per Cent Paid by Mail**

Approximately 70 per cent of all bills are paid by mail. This arrangement is encouraged, as it alleviates congestion in the City Hall. Bills are receipted officially and the cashiers' stubs deposited in locked boxes. Twice each day an auditing clerk removes these stubs from the cashiers' cages and balances their receipts. The cashier submits his statement of accounts collected in bulk, and this is later verified through the break-down of the cashiers' stubs by collection district and revenue classification. When the complete balance for all cashiers has been effected, a master sheet for the day's entire collection is prepared by the Head Cashier, of which one copy passes to the general bookkeeper, a second copy to the City Auditor, and a third copy is retained.

The cashiers' stubs are then passed to the accounts receivable unit clerk, who removes the corresponding accounting record from the

unpaid files, in accordance with the cashiers' stubs. When they all have been removed, they are totaled on a listing machine, and this amount must agree with the auditing clerk's distribution of the cashiers' receipts. When all the unit clerks have balanced, then the verification of the auditing clerk's distribution of the cashiers receipts is completed. The stubs are then stamped "Paid" and filed in paid receptacle. This procedure leaves us at all times only unpaid accounts with which to deal. For example, bills rendered on the 4th, and payable on the 14th, will have accounting records only on approximately 2,000 of the original 11,000 bills rendered, in the unpaid file, on the 17th. The 9,000 delinquent notices on accounts paid are destroyed, while the 2,000 notices for unpaid accounts are mailed in the manner described previously. At the expiration of the second date of payment, in this instance the 25th of the month, there are approximately 300 accounts left. Instead of being required to make a search of either ledger or register cards to abstract these 300 accounts out of a total of 11,000, we automatically have them before us for the preparation of the shut-off bills. Three copies of these delinquent bills are prepared: blue, gray and white. The blue copy is retained in the cashier's office, that he may have a record of all past due accounts. The gray and white copies are passed to the turnkey foreman who gives the white bill to the turnkey for collection and keeps the gray copy as a follow-up. As the turnkeys report their collections or shut-offs, the turnkey foreman matches up his gray copy and turns both over to the Cashiers, who remove the blue copy. As a past due bill is paid to the Cashier he stamps the blue copy "paid" and sends it to the turnkey foreman who immediately recalls the white bill. In this manner, double payments or unnecessary calls are eliminated. These bills are all paid up in five days and enable the Commercial Superintendent to keep a close supervision on the collections.

It will be noted that all control entries go through the general bookkeeper; the amount of the bills, the amount of cash credited, and any allowances are authorized only by the Commercial Superintendent. These few adjustments are prepared in duplicate, one copy going to the general bookkeeper and one copy to the unit clerk for posting to the accounting record. This operation prevents any collusion and enables positive balances to be effected at any time.

It might appear that, subconsciously, an attempt has been made to prove the stub plan of accounting superior to all others. We are

to be forgiven if this paper creates that impression. The only thing we will say is that it has been so successful in the Cincinnati Water Works that many other cities have adopted the plan. The advantages to be gained from the foregoing procedure might be listed as follows:

1. The prompt mailing of notices of excessive consumption to consumers, and the follow-up to see that all leaks have been repaired.
2. The notice to the Meter Division of "stuck" meters.
3. Billing in cycles of ten-day intervals, which eliminates peak loads and permits of a small personnel, which, equipped with modern mechanical means of billing and accounting, takes care of a tremendous additional load, due to potential growth in the number of consumers. Here we might illustrate that 88,000 consumers had meter reading, billing, accounting and collecting handled by 58 employees in 1930. Today, 102,000 accounts are handled through the same procedure by 59 employees. This increase necessitated the employment of only one additional meter reader.
4. Facility with which delinquent bills may be prepared for collection or shut-off purposes.
5. Flexibility of the system for payment of water bills in installments, with accurate follow-up to insure such payments.
6. Facility of balancing at the close of the collection period, so that all accounts have either been paid or shut off.
7. Expediency in handling public relations, by granting almost any reasonable request to meet with the consumer's financial status.

In conclusion, it might be noted that this system has been the means of such large savings that in the eight years of its operation two substantial reductions in rates have been made, saving consumers approximately \$565,000 per year.



## The Aging of Reservoir Waters

*By Lee T. Purcell*

IT IS a well known fact that the quality of water impounded in large reservoirs changes markedly in chemical and physical characteristics during storage. These changes are mostly beneficial but there are many harmful changes which take place particularly in the early years following a reservoir's initial filling. Fortunately as time passes on, nature gradually irons out these early defects. This natural process of ironing out, let us call "aging of reservoirs."

The phenomenon of "reservoir aging" is vital to the sanitary engineer, particularly so if the purity of his water supply is dependent to a large degree on the purifying forces of nature. In the early days when large and small communities alike derived their water from nearby flowing streams or rivers, reservoir aging was little thought of. But the trend has been growing away from nearby flowing streams to upland impounded supplies and more emphasis has been placed on the study of waters in storage.

The quality of an impounded water and the period of aging of that water depend on certain factors which must be taken into account in the location of a reservoir site. These factors are: (1) topography of the drainage area and area to be flooded; (2) storage ratio; and (3) reservoir depth and area.

If the slopes of the drainage area are rugged and steep, the percentage of land under cultivation and swamp areas low, and the land to be flooded is grubbed and stripped of trees, vegetation and top soil, ideal conditions for rapid reservoir aging may exist. When such conditions prevail, it is possible to predict with a degree of accuracy the ultimate color and turbidity of water stored and the

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length of time necessary for aging. The sanitary engineer, knowing the nature of the underlying soil and rocks may prognosticate the ultimate mineral character of the water stored. Sanitary surveys of the drainage area permit him to draw conclusions as to the bacteriological quality to be expected.

However these predictions are based on ultimate conditions when the reservoir has "aged" and may not be those actually existent during the aging period. What is a normal period for reservoir aging? This question may be answered only after there is a full understanding of the various factors previously mentioned and a knowledge of the general limnological conditions as they prevail in the reservoir. In order to comprehend these limnological factors, it is felt advisable here to review briefly the phenomena of fall and spring turnover, winter and summer stagnation and their effects on quality of water stored.

In brief, a reservoir turnover occurs when the upper layers of water become heavier than the deeper waters and settle to the bottom displacing the bottom water to the upper levels. For example, during the winter seasons, when reservoirs freeze over, the temperature of the water beneath the ice may vary from a point just above freezing in the layer immediately adjacent to the ice to a point a few degrees above freezing at the bottom. It is of course generally known that water has its maximum density at 4°C. (39.2°F.) and that water below and above 4°C. is therefore lighter than at four degrees (see table 1). Consequently when the ice disappears from the reservoir, the surface water then being exposed to the warmer air above warms up to 4°C. and sinks to the deeper regions and displaces the bottom water. Following this phenomenon, which is called the spring turnover, the reservoir begins to stratify with the temperatures varying from a minimum at the bottom to a maximum at the surface. As the summer season progresses the reservoir becomes definitely stratified with the surface water temperature reaching nearly 25°C. and the bottom water, depending on the depth of the reservoir, at approximately 8 to 10°C. When the cool fall air chills the surface waters to a point which is colder than the bottom water, thermal circulation takes place causing a continuous displacement of deeper waters by the heavier surface water. This is called the fall turnover. Following the fall turnover there may be a period of winter stratification or winter stagnation and the cycle retraces its seasonal path.

In theory the fall and spring turnovers occur when the temperature

of maximum density is reached; however, in practice the fall turnover may be expected in this climate when the temperature of the surface water reaches approximately 8° to 10°C. (46.4° to 50°F.). In this latitude, reservoirs are little affected by winter stratification and spring turnover and suffice it to say that in most cases they pass unnoticed except for possibly a slight increase in organic matter on chlorinator screens, etc. Waters in warmer climates have been reported as stratifying with only a fluctuation of a very few degrees. John R. Downes (1) has reported that shallow reservoirs (only 9, 12 and 25 ft. deep) in Panama have experienced all of the effects of summer stagnation and turnovers that are experienced by deeper

TABLE 1  
*Density of Water and Thermal Resistance to Mixture*  
(After Birge)

TEMPERATURE		DENSITY	DIFFERENCE IN DENSITY FOR 1°C.	TEMPERATURE		DENSITY	DIFFERENCE IN DENSITY FOR 1°C.
°C.	°F.			°C.	°F.		
0	32	0.999868	+0.000059	11		0.999632	-0.000107
1		0.999927	+0.000041	12		0.999525	-0.000121
2		0.999968	+0.000024	13		0.999404	-0.000133
3		0.999992	+0.000008	14		0.999271	-0.000145
4	39.2	1.000000	-0.000008	15	59	0.999126	-0.000156
5	41	0.999992	-0.000024	16		0.998970	-0.000169
6		0.999968	-0.000039	17		0.998801	-0.000179
7		0.999929	-0.000053	18		0.998622	-0.000190
8		0.999876	-0.000068	19		0.998432	-0.000202
9		0.999808	-0.000081	20	68	0.998230	-0.000211
10	50	0.999727	-0.000095				

reservoirs in this climate. In these reservoirs, turnovers must of necessity occur when the surface waters are cooled to a temperature below that of the deeper waters. In these cases a temperature of 4°C., maximum density, never is reached.

The early work of Prof. George C. Whipple (2) has been the text book guide in limnology. Prof. Whipple has described limnology as "that branch of science which deals with lakes and ponds—their geology, physiography, physics, chemistry and biology. . . ." *Thermocline Studies* by Dr. Frank E. Hale (3) in 1916 and others are worthy of note. It is felt that there has been a paucity of data published on these subjects in more recent years and for that reason the scope of

this paper will include some of the data collected by these earlier writers with more recent data collected on newly constructed reservoirs.

Large impounding reservoirs are not frequently constructed and as is very often the case a period of 25 years or more may elapse between these developments in any one area. Consequently it is felt that the analytical data collected from a drainage area before and during a reservoir's construction and at regular intervals since its completion over a period of years may be of some value. The purpose, therefore, of this paper is to fill this gap in a modest way and to contribute even a small part to the work published in earlier years. It is felt that with such data available, the water works engineer may more intelligently predict the quality of water he may anticipate rather than be caught unawares by the apparent "acting up" of a new reservoir.

The Wanaque Reservoir, the largest in the State of New Jersey, a 100 m.g.d. supply, constructed in Northern New Jersey and developed and operated by the North Jersey District Water Supply Commission for the Metropolitan New Jersey area, has been an excellent proving ground for limnological studies. This paper will, for the most part, be made up of data from this reservoir. Construction of the reservoir was begun in 1920 and in 1928 the gates were closed for filling. In March, 1929, the reservoir was filled for the first time and one year later water was first delivered to the consumers. Analytical data were collected from the streams on this drainage area since 1923 and from the reservoir during construction and since completion.

Studies of the characteristics of water at various depths in the reservoir have been carried out following a biweekly schedule of sampling.

Seasonal variations in temperatures at various depths are given in table 2. Three seasonal periods were taken to show the variation in the spring of the year when stratification is in its early stage, during August when stagnation is well advanced, and a typical temperature relation following the fall overturn.

Records were kept of the dissolved gases, microscopic organisms and temperature changes in the Wanaque Reservoir at various depths during 1930, the first year of operation.

Samples collected on April 1, 1930 indicated that the reservoir was in a state of vertical circulation with the temperature constant throughout its entire depth and below the point of maximum density

(4°C.). The dissolved oxygen content was practically at 100 per cent saturation from the surface to the bottom.

On May 28, 1930 evidence of summer stratification was noted. The thermocline or transition zone (that zone showing maximum temperature change) was found between the 20- and 40-foot depths. The dissolved oxygen reduction was notable beginning at 20-foot depth and reached a minimum of 35 per cent in the bottom sample.

Samples collected on July 18, 1930 indicated a definite stratification with the dissolved oxygen approaching zero in the bottom sample and a concentration of micro-organisms in the upper 20 ft. with the maximum number at the 20-foot level. This was caused by the settling out of organisms killed by copper sulfate to the top of the

TABLE 2  
*Temperature Studies on the Wanaque Reservoir, 1938*

DEPTH	MAY 19	AUGUST 1	NOVEMBER 29
	°C.	°C.	°C.
Surface	14.9	25.4	8.1
10 ft.	14.5	25.3	8.1
20 ft.	13.6	24.8	8.1
30 ft.	10.8	21.2	8.1
35 ft.	—	20.1	—
40 ft.	9.0	17.0	8.0
50 ft.	8.4	10.4	8.0
60 ft.	7.8	9.0	7.9
70 ft.	7.7	8.5	—
72 ft.	—	—	7.9
75 ft.	—	8.1	—
77 ft.	7.5	—	—

transition zone. The carbon dioxide increased gradually to 25 p.p.m. at the bottom.

Samples collected August 18, 1930 indicated a narrowing of the transition zone between the 25- and 30-foot level. Below the 25-foot level there was practically no oxygen.

On September 17, 1930 the dissolved oxygen had increased at the 25-foot depth. This clearly indicates thermal circulation and circulation due to wind action and not microscopic growths. The transition zone was in the 30-to-40-foot depth. The number of micro-organisms was at a maximum at the 10-foot depth. The carbon dioxide content increased between the 20-foot depth and the

bottom of the transition zone. It remained nearly constant for the next 10-foot depth and then increased notably at the bottom. There were practically no micro-organisms below the transition zone.

On October 10, 1930 the curves indicated vertical circulation to a 35-foot depth. Wind action and thermal circulation replenished oxygen in the 30-foot depth. The carbon dioxide content remained practically the same as of September 17. It is felt that blowing water at the rate of 20 m.g.d. from the bottom blow-off at this time prevented the carbon dioxide from reaching a much higher point.

Samples collected on November 12, 1930 indicated that the fall turnover had occurred at a temperature between 9 and 10°C. This is indicated by a nearly constant temperature, carbon dioxide and dissolved oxygen content for the entire reservoir depth.

Organic decomposition was so advanced by August 18, that there was little oxygen present below the 25-foot level. This gave rise to anaerobic bacterial activity in the lower regions resulting in the breaking up of sulfates and liberation of hydrogen sulfide. This condition may be expected during the first few years in a newly constructed deep impounding reservoir. It is needless to state of course that in the presence of hydrogen sulfide the chlorine demand is increased to a critical point. For the first three years, hydrogen sulfide was present in decreasing amounts in the deeper water. The maximum hydrogen sulfide content recorded was 1.13 p.p.m. on September 17, 1930 in the bottom sample collected 77 ft. below the surface, although there is reason to believe that in 1928 and 1929 the hydrogen sulfide content was much higher. Following the disappearance of hydrogen sulfide there was no absolute depletion of dissolved oxygen. It would indicate therefore that in the Wanaque Reservoir and probably in most large reservoirs that there is little or no dissolved oxygen for a period of 3 or 4 months prior to the fall turnover in the deeper water during the first three or four years after initial filling and that this zone of zero oxygen may reach upward even into the thermocline. In some instances dead microscopic organisms which settle out or concentrate in the transition zone may decompose and deplete the oxygen. A luxuriant green or blue green algae growth has been known to increase the dissolved oxygen content above point of saturation in the surface waters.

Thermocline studies of the Wanaque Reservoir after nine years of reservoir aging revealed that in general thermal conditions were

similar. There was a concentration of micro-organisms in the upper regions with occasional concentrations in the layer immediately above the thermocline, and there was uniform distribution from the surface to the bottom at turnover periods. There was a marked reduction in carbon dioxide and an improvement in dissolved oxygen.

Samples collected April 25, 1938 indicated the beginning of summer stratification. The carbon dioxide and dissolved oxygen contents had not as yet been affected. The thermocline was located between the 30- and 40-foot depths. The micro-organisms were fairly evenly distributed through the reservoir depth.

The samples collected on May 19, 1938 showed that the carbon dioxide had not as yet increased in the deeper waters but below the 20-foot level the dissolved oxygen had begun to decrease. The bottom samples contained 87.7 per cent oxygen as compared with only 33.3 per cent on May 18, 1930. This was a marked improvement and indicated very definitely a slowing up of organic decomposition in the bottom water, a notable mark of improvement due to aging.

On July 6, 1938 the transition zone continued between the 20- and 40-foot depths. The dissolved oxygen content dropped 25 per cent within this zone. Above the thermocline the dissolved oxygen remained at the saturation point while below the thermocline the dissolved oxygen was constant at about 75 per cent saturation. Below the thermocline, the carbon dioxide content showed evidence of increasing. The carbon dioxide content in the bottom water on this date was approximately 40 per cent lower than at the same depth on July 10, 1930. The dissolved oxygen content in the top 20 feet of water was practically the same on July 6, 1938 as on July 10, 1930. Within and below the transition zone on July 6, 1938 the dissolved oxygen was notably greater than on the comparable 1930 date when the dissolved oxygen in the bottom water was less than 20 per cent saturated.

On August 21, 1938 the dissolved oxygen dropped noticeably in the 10-foot zone above the thermocline and decreased to 50 per cent saturation in the transition zone. It remained practically constant between the 30- and the 60-foot depth and in the bottom water dropped to a minimum of 30 per cent for the year. On this date the transition zone extended from a point 20 ft. below the surface to a point 50 ft. below the surface. The dissolved oxygen content on this

date in the 20- and 30-foot depth varied between 90 and 50 per cent saturation as compared with a practical zero oxygen zone on August 18, 1930 at and below the 25-foot depth.

Samples collected subsequent to August 21 indicated that the carbon dioxide content in the bottom water had reached a peak on that date and remained practically constant to the turnover period when it again returned to normal. The dissolved oxygen content remained fairly constant in the bottom water from August 21 to the turnover period but decreased markedly within and below the transition zone. This indicates decomposition of suspended organic matter which had settled to the zone of greater viscosity or greater density.

Curves plotted for the samples collected November 29, 1938 indicated a return to normalcy following the turnover.

By frequent depth sampling and by weighting the various characteristics for each ten-foot depth zone, the analyst may predict with a fair degree of accuracy the quality of water to be expected immediately following the turnover.

Figure 1 shows the relationship between the carbon dioxide and dissolved oxygen contents in the Wanaque Reservoir since 1929. The carbon dioxide curve indicates a fairly constant value during the winter months at a point near normal saturation (approximately 2 p.p.m.). This is further proof of the practicable absence of winter stagnation in this reservoir. Beginning in the month of June there is a gradual increase to a peak at the climax of summer stagnation period. The curve indicates clearly that the maximum carbon dioxide content occurred in the fall of 1931. However in all probability the peak would have been reached one year earlier except for the fact herein before stated that 20 m.g. of water daily were blown off from the reservoir bottom. This gives rise to the argument whether or not it is best to blow off bottom waters of new reservoirs. The arguments in favor may be: reduction in carbon dioxide and hydrogen sulfide with coincident reduction of other products of decomposition. On the other hand, the strongest argument against blowing off bottom water may be that natural coagulation, which is assisted by the hydrolyzing of mineral salts, is impeded and therefore the reservoir requires a longer period following the turnover to return to normalcy.

The carbon dioxide curve also indicates that there was a gradual reduction in carbon dioxide during the first five years after which the

carbon dioxide remained fairly constant. Since lime treatment is commonly used in neutralizing free carbon dioxide in corrosion prevention, the value of natural carbon dioxide reduction is of prime importance.

The dissolved oxygen curve indicated that, coincident with peak carbon dioxide, there is a minimum of dissolved oxygen. The dissolved oxygen remains at a point near saturation during the winter and spring months and beginning in May there is a gradual decrease in oxygen to a minimum when summer stratification is most evident.

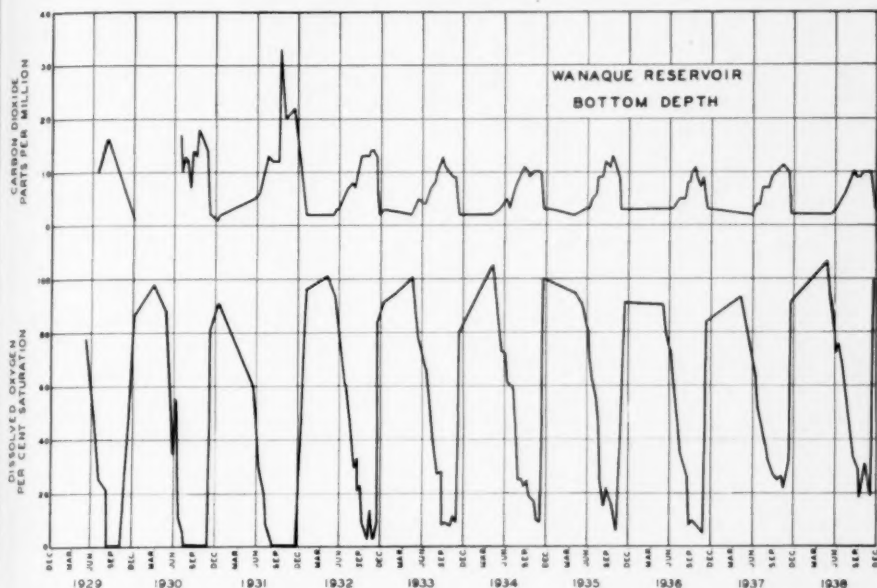


FIG. 1. Relationship Between Carbon Dioxide and Dissolved Oxygen

This curve also shows that during the first three years there was a period of nearly four months of absolute oxygen depletion at the bottom of the reservoir and for the next five years there was an approximate average of 10 per cent saturation. For the past two years there has been an average of at least 20 per cent saturation in the bottom water.

### Color

Color or lack of color is an important attribute of a water supply. It ranges from zero in well waters to a weak tea or even strong tea

color in waters collected from or over swampy areas. Color is caused by the solution of organic matter from the drainage area or the bottom ooze of reservoirs. It may also be caused by mineral salts in solution. In large reservoirs, as has been previously stated, organic decomposition takes place in the bottom waters during the summer stratification period causing an increase in carbonic acid and resulting in the solution of organic matter and mineral salts such as iron and manganese. As the reservoir becomes aged, this rate of decomposition decreases with consequent decrease in bottom water color. Bleaching of surface water greatly improves the color.

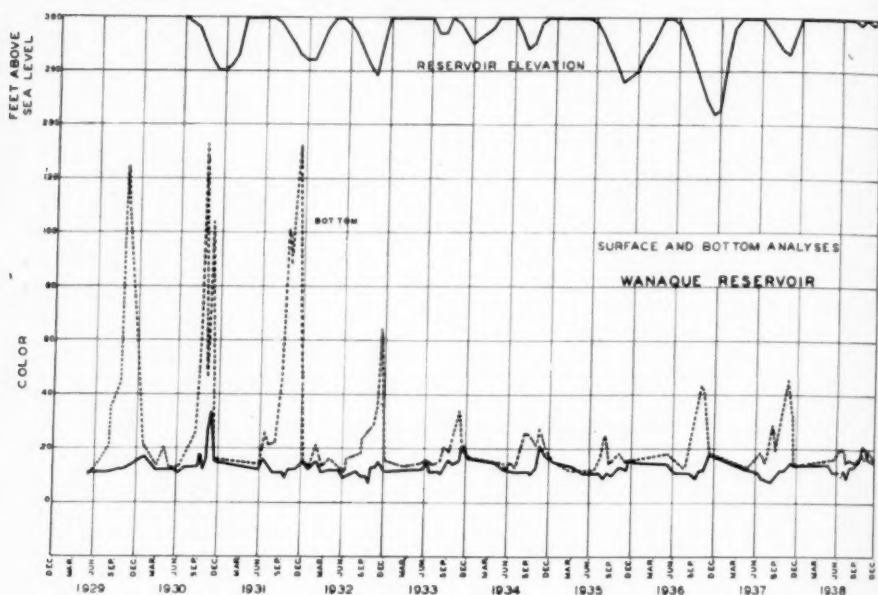


FIG. 2. Trend of Color

Figure 2 indicates the great increase in color in the bottom waters of the Wanaque Reservoir at the period immediately before the turnover, during the first three years after filling the new reservoir. In 1932 the maximum color in the bottom water was approximately 50 per cent of that prevailing during the fall months of 1929, 1930 and 1931. In 1933 there was a further reduction of 50 per cent over 1932 and in 1934 and 1935 the color gradually reduced to a minimum for this season. This curve indicates also that there was no color reduction in the bottom water during the first three years. It is

evident too that the bottom water color increased in 1936 and 1937 and returned to a minimum again in 1938. It is felt that the successive low elevation in 1935 and 1936 during the fall season and the rapid refilling in 1936 increased the organic load causing a higher bottom water color in 1936 and 1937. The return to normal in bottom water color in 1938, when the reservoir had been near the overflow elevation during practically the entire year, tends to prove that factors such as drawdown and runoff may offset the benefits of aging. The surface water color attained a peak generally at the

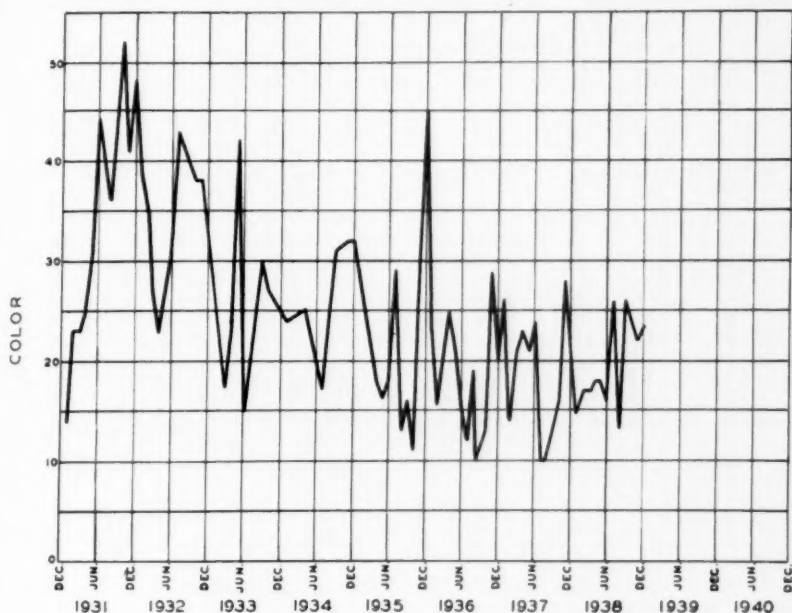


FIG. 3. Color Trend in Cupsaw Lake

turnover period. It is interesting to note that the reservoir surface water had a higher color than the bottom water on October 1, 1938. This of course was directly caused by the heavy rains of September 19, 20 and 21, followed by the strong winds that reached hurricane proportions on September 21, 1938.

Figure 3 indicates the color reduction due to aging of Cupsaw Lake, Ringwood, New Jersey. Cupsaw was developed as a lake resort in 1931. It may be classed as an unused reservoir. It has a drainage area of 5.4 sq.mi., a storage capacity of 87 million gallons

1926 and may also be classified as an unused reservoir. It has a drainage area of 5.3 sq.mi.—practically the same as Cupsaw Lake. The storage capacity of 200 million gallons is more than twice as great as Cupsaw Lake. The storage ratio is approximately 0.10. The larger storage capacity and storage ratio permits a greater color reduction due to bleaching and the influence of heavy rains is less noticeable, though still quite marked as compared with the Wanaque reservoir having a storage ratio of approximately 1.0. In Erskine Lake (constructed similarly to Cupsaw Lake) the color peak was reduced 30 per cent during the second year and remained at the same point during the third year. The fifth year showed more than a

50 per cent reduction and in the eighth year, 1934, the color had reached a balanced reduction of 80 per cent of the first year peak. During 1935 and 1936 the peak color was raised by heavy rains and in 1937 and the early part of 1938 it again returned to normalcy at a value near that of 1934.

Figure 5, the Greenwood Lake curve, indicates the color trend in a lake which is 103 years old. Greenwood Lake, an unused reservoir, has a storage of six billion gallons and a drainage area of 27.1 sq.mi. The storage ratio is approximately .60 or very nearly equal to half of the storage ratio of the Wanaque Reservoir. The color curve for

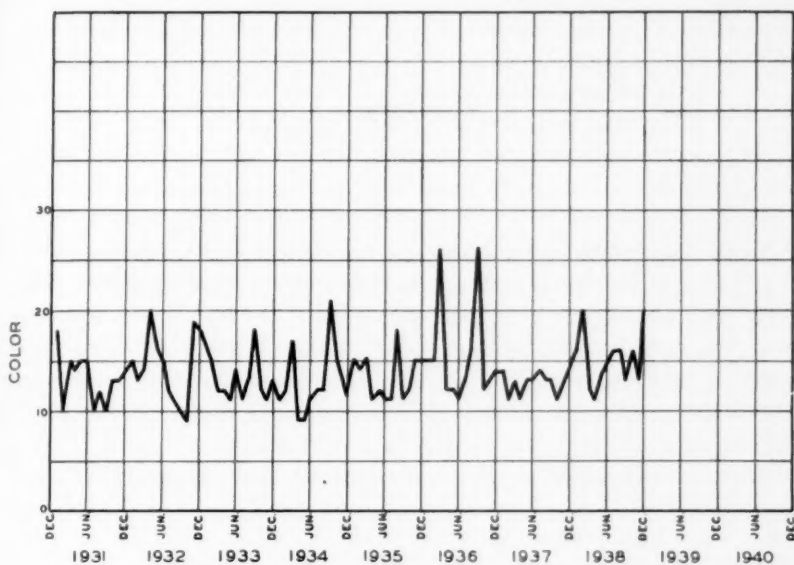


FIG. 5. Color Trend in Greenwood Lake

this lake is fairly uniform, averaging somewhat less than 15. It is quite significant that the color peaks may not be directly attributed to turnovers. However this does not mean that stagnation does not occur in older lakes but rather that the effects of turnover are less noticeable with age.

In the location of a new reservoir, the consulting engineer very frequently is faced with the task of predicting the color of water to be found after the reservoir has been completed.

Mr. Weston, who is a discussor of this paper and under whose supervision the early laboratory work from 1923 to 1932 was carried

out, was faced with the problem of predicting an ultimate color for the Wanaque Reservoir. The average apparent color in this reservoir during 1937 was 13 as compared with his prediction of 12.

A reservoir water after it has aged should possess a color equal to the weighted color of all of the streams entering this reservoir less a percentage reduction due to storage. In the Wanaque Reservoir this percentage reduction of color, due principally to bleaching, has been found to average 21 per cent.

The weighted color of any stream in a reservoir catchment area is equal to the average color of that stream  $\times$  the runoff of that stream divided by the total drainage area runoff.

Where: B = average color

A = weighted color

C = stream runoff in m.g.

D = total runoff of all streams in m.g.

$$A = \frac{B \times C}{D}$$

Table 3 shows the average yearly color of each of the streams on the Wanaque Reservoir drainage area from 1931 through 1938. The table also shows the yearly weighted color for the entire drainage area, the actual yearly average color for the reservoir water and the yearly percentage reduction in color for the years 1931-38.

Whipple in *The Microscopy of Drinking Water* states: "A series of experiments made at the Boston Water Works by exposing bottles of high-colored water to direct sunlight for known periods showed

TABLE 3  
Color Data

YEAR	WEIGHTED COLOR, WANAQUE WATERSHED	ACTUAL COLOR, WANAQUE RESERVOIR	% REDUCTION
1931	17.2	15	12.8
1932	17.7	13	26.5
1933	18.6	15	19.4
1934	18.6	15	19.4
1935	17.6	12	31.8
1936	16.5	14	15.2
1937	16.8	13	22.6
1938	18.9	15	20.6

Average Percentage Reduction—21 per cent.

that during 100 hours of bright sunlight the color was reduced 20 per cent and that with sufficient exposure all the color might be removed."

### Turbidity

Turbidity may be caused by suspended matter such as silt, organic matter and mineral salts. In the earlier years of a reservoir's existence an opalescent turbidity is frequently found in the deeper water which may extend well up into the transition zone. This opalescent turbidity is caused by a colloidal suspension of ferrous iron. Following a turnover there is sufficient oxygen to oxidize this ferrous to ferric iron and to reprecipitate it. The precipitation of iron acts as a coagulant in a reservoir and assists the reservoir in a more rapid return to normal.

### Nitrogen

Nitrogen may be considered as being made up of suspended organic nitrogen and soluble (including colloidal) nitrogen. Organic nitrogen is greater in the surface waters and is derived principally from suspended microscopic organisms. Soluble nitrogen is the nitrogen extracted from the bottom ooze of reservoirs.

TABLE 4  
*Wanaque Reservoir Yearly Nitrogen Averages*

P.P.M. NITROGEN	1932	1933	1934	1935	1936	1937	1938
Free ammonia.....	.039	.016	.009	.013	.022	.014	.009
Albuminoid ammonia.....	.099	.090	.070	.083	.064	.070	.073
Nitrites.....	.005	.003	.000	.002	.003	.003	.001
Nitrates.....	.04	.03	.01	.05	.03	.02	.02

Table 4 gives the average nitrogen values in p.p.m. for the Wanaque Reservoir since 1932. This table would indicate that the free ammonia varies from year to year. The albuminoid ammonia variation is much less than the free ammonia, and the variation in the nitrites and nitrates is practically nil.

The seasonal variations of nitrogen in its various forms may be great. In the Wanaque Reservoir the seasonal variations in the nitrites and nitrates since 1932 have been slight. The free ammonia nitrogen was greatest in the winter and fall months with the summer months showing the least free ammonia, except in 1932 when during June, July and August, the free ammonia was at its maximum.

The albuminoid ammonia variation somewhat paralleled the free ammonia variation.

The total nitrogen (sum of free and albuminoid ammonia, nitrite and nitrate, nitrogens) reached maximums during the winter and fall months and minimums in the summer months except in 1932 when the summer months of July and August exhibited maximums (see table 5).

The average yearly total nitrogen values for Greenwood Lake are given in table 6 from 1923 to and including 1938.

### Iron and Manganese

Iron and manganese occur in reservoir streams and in combination with organic matter in the bottom ooze of many reservoirs. During

TABLE 5  
*Wanaque Reservoir Monthly Variations in Total Nitrogen*  
(As Nitrogen in p.p.m.)

	1932	1933	1934	1935	1936	1937	1938
January.....	—	—	.126	.106	.126	.120	.149
February.....	—	.157	.141	.111	.124	.107	.101
March.....	.171	.181	.146	.106	.162	.094	.085
April.....	.130	.128	.159	.099	.122	.121	.111
May.....	.143	.137	.120	.062	.105	.103	.109
June.....	.178	.140	.144	.060	.115	.112	.081
July.....	.231	.129	.100	.060	.070	.090	.087
August.....	.197	.079	.072	.064	.069	.070	.087
September.....	.151	.116	.154	.072	.102	.088	.081
October.....	.177	.127	—	.092	.096	.128	.083
November.....	.144	.141	.097	.123	.118	.106	.105
December.....	.154	.126	.105	.123	.141	.105	.100

the early years of a reservoir's existence, iron and manganese may reach concentrations great enough to cause serious complaints. The iron and manganese salts exist in a more or less oxidized condition in the bottom ooze of a reservoir during the winter and spring months in this climate. As the period of summer stratification progresses and organic decomposition increases the iron and manganese compounds give up part of their oxygen and are changed from a fairly insoluble ferric and manganic to the more soluble ferrous and manganous state. Carbon dioxide—a product of decomposition—places these minerals in a colloidal state as ferrous and manganous bicar-

bonates. Upon oxidation of the lower waters which takes place at the turnover period the minerals become oxidized once again and precipitate to the bottom. However, many complaints in the delivered water during the summer and fall months may be attributed to these salts.

Iron being more readily soluble and also more readily oxidized shows a more rapid reduction on aging. Figure 6 indicates an iron content of nearly 3.0 p.p.m. in the bottom sample just previous to the turnover in 1931, two years after the reservoir was first filled. In November, 1932, one year later, the iron content dropped to 1.18 p.p.m. at the same depth. This was a reduction of 60 per cent. There was a similar percentage reduction during the next two years. A minimum of 0.03 p.p.m. for the bottom water was reached in 1935.

TABLE 6  
*Greenwood Lake Yearly Average Total Nitrogen in p.p.m.*

YEAR	TOTAL NITROGEN	YEAR	TOTAL NITROGEN
1923 & 1924	.177	1932	.209
1925	.133	1933	.184
1926	.131	1934	.190
1927	.152	1935	.148
1928	.116	1936	.171
1929	.157	1937	.246*
1930	.184	1938	.161
1931	.185		

\* High figure due to albuminoid ammonia values approximately 50 per cent greater than normal.

This would indicate that in seven years the bottom water of the reservoir had probably become of "age" as far as iron content was concerned. During the fall of 1936 and 1937 the bottom water iron content increased to a point greater than in 1933, 1934 and 1935. In 1937 the peak bottom water iron content was even greater than in 1932. In 1938 it once again returned to a normal point below the 1935 level. The 1936 and 1937 peaks were probably caused by the rapid filling of the reservoir during 1935 and 1936 after a maximum draw down to 280.95 ft. above sea-level.

The surface water iron curve showed definite peaks at the turnover period although these peaks were only slightly above the normal iron content.

Figure 6 also shows the relationship between the manganese content of the surface and bottom water. The peak of the bottom water curve is again found at the climax of the stratification period immediately prior to the turnover. In 1931 the maximum manganese content in the bottom water was 8.75 p.p.m. There was a marked improvement of nearly 65 per cent in 1932, and a continued improvement throughout 1934. In 1935 and 1936 there was a return to the 1933 peak. There was a gradual improvement in the fall of 1937 and 1938. The surface water showed definite peaks imme-

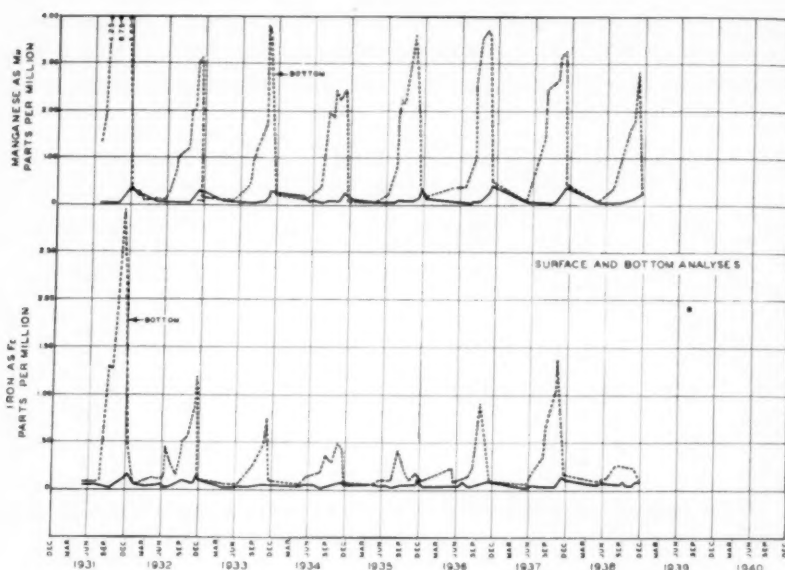


FIG. 6. Iron and Manganese in Wanaque Reservoir

diately following the overturn in each case. The higher surface water manganese peak in 1936 was caused by the turnover occurring during the low reservoir storage or maximum drawdown.

The iron and manganese contents of a reservoir should be equal to the weighted iron and manganese of streams entering the reservoir when that reservoir has ceased dissolving these salts from its original bottom soil and from the organic matter precipitated from the water itself. It is reasonable to feel that this condition may be reached more quickly in the case of iron than with manganese. Table 7 gives the weighted iron content by years for each of the streams

entering the Wanaque Reservoir since 1935, the actual raw water iron content of the reservoir, and the percentage reduction.

The fact that the reservoir water iron content is lower than the weighted iron content of the streams entering the reservoir would indicate that precipitation of iron must take place in the reservoir coves at the mouth of each stream. There is reason to believe that such is the case.

A very definite pickup in manganese during storage is shown by the data in table 8. This is an indication that from a manganese

TABLE 7  
*Iron Content in p.p.m. of Wanaque Reservoir*

	WEIGHTED (STREAMS)	RESERVOIR WATER	% REDUCTION
		<i>p.p.m. Fe</i>	
1933	—	.07	—
1934	—	.04	—
1935	.11	.04	63.7
1936	.09	.06	33.3
1937	.08	.06	25.0
1938	.12	.05	58.3

TABLE 8  
*Manganese Content in p.p.m. of Wanaque Reservoir*

	WEIGHTED STREAMS	RESERVOIR WATER
1933	—	.17
1934	—	.13
1935	.05	.11
1936	.04	.15
1937	.04	.16
1938	.03	.11

content angle the reservoir has not yet received the maximum benefits of aging, although a vast improvement is shown. The average manganese content showed a big improvement since 1931 when a peak of 8.75 p.p.m. was found in the bottom water. The manganese content of the bottom water dropped to a low maximum in 1934 and returned in the fall of 1935 to a peak nearly as high as in 1933. Since 1936 there has been a downward trend in the fall peaks. The surface water samples showed definite peaks at the turnover periods.

Of course the elevation of the reservoir or the amount of storage at the turnover has a great bearing on the manganese content of the reservoir water at the turnover.

### Hydrogen Sulfide

In the process of putrefaction and organic decomposition in the bottom of reservoirs, hydrogen sulfide is evolved in the early years of a reservoir's existence. Hydrogen sulfide is generated by the bacterial reduction of sulfates. During the summer and early fall of 1928 and 1929, when the Wanaque reservoir was filling, there was 0.93 p.p.m. hydrogen sulfide in the bottom water. On September 17, 1930 there was 1.13 p.p.m. and on October 8, 1931 there was 0.63 p.p.m. At no time since 1931 has hydrogen sulfide been found. It is needless to say that the presence of hydrogen sulfide presents a disagreeable odor condition in a public water supply and also creates an immense chlorine demand. In 1928 and 1929 there was a slight odor of hydrogen sulfide detected coming from the surface of the reservoir. The reservoir was not in use at the time. However this was an indication that the gas was building up from the bottom of the reservoir through the transition zone even to the surface. Only occasional analyses of hydrogen sulfide were made prior to October 10, 1929, and it is felt possible that a higher peak of hydrogen sulfide may have been attained than that noted. During this same period in 1930 the hydrogen sulfide content reached a peak of 1.13 p.p.m. in the bottom water and was found well up into the 20-foot zone. In 1931 the amount dropped sharply to a peak of .63 p.p.m. Since that time there has been no hydrogen sulfide at the reservoir bottom and of course no oxygen depletion in that zone. It is evident therefore that the presence of hydrogen sulfide should be expected in a large deep reservoir during the first few years of its existence.

### Microscopic Organisms

The numbers and species of microscopic organisms in a reservoir vary with the season of the year, the intensity of light, the quantity of food materials, the size of the reservoir and the nature of the shore line.

C. A. Kofoid (4) in 1923 drew a close analogy between land plants and animals and aquatic plants and animals. "This analogy is here quoted in part. "The land and water share alike the capacity of production of an annual crop of vegetation and of animal life feeding

thereon, dependent upon the seasonal course of solar illumination. . . . In the same way the permanent bodies of fresh water, including reservoirs, which man constructs, also produce . . . annual returns in fish. . . .

"This plant life of water, however, differs from that of the land in not building up large growths, such as the grasses, cereals, alfalfa and trees, but in being composed almost wholly of low and simple, primitive forms of vegetable life known as diatoms, desmids, dinoflagellates and algae. These are photosynthetic organisms which multiply with astounding rapidity and produce throughout the year successive crops of microscopic vegetation. These are the primitive food supply of the animal world of the water, which in turn is in part itself microscopic and fluctuates in quick response to the ever-changing quantities of plant life. These quantities of the plant life are produced in rapidly succeeding waves or pulses of production throughout the year and owe their changing dimensions to the fluctuations in light as influenced by the length of the day, the amount of sunlight and primarily the total amount of radiant solar energy which reaches the water. The time factor, the chemical substances in solution, and the temperature modify the amount and the rate at which growth proceeds and thus determine the tonnage per acre of the reservoir. . . .

"The seasonal sequences of the changes of the vegetation and the accompanying animal life in the reservoir are not unlike those of the land in certain striking particulars. In the first place periods of rapid growth occur at the same season of the year in both regions. When the elms and maples are leafing out in early spring in April on the shore of the reservoir, the water itself is flushed with the fresh growth of diatoms and algae. These growths occur year after year with the same regularity as does the bursting of the buds of the maple or the catkins of the willow, and are exquisitely timed to certain sum totals of heat which may be measured by adding up the total heat of each day as the season progresses. In some years the total heat of each day attains the flushing level earlier than others and we have an early spring both on land and in the reservoirs.

"The autumn season on land often brings a recurrence of a second and minor period of renewed growth of vegetation, and reservoirs repeat this with an autumnal flush or growth of diatoms, similar to that of the spring, but of lesser volume. The winter is a season of lessened production, but of different species as a whole from that of

the summer, and the summer months a season of recurrent minor flushes not unlike the successive crops of alfalfa that may be reaped from the meadow lands. These, however, do not equal in volume the production of the spring and fall as a general rule."

Generally speaking diatoms are more commonly encountered in fall, winter and spring months. Chlorophyceae (green algae) are generally found in the early summer and Cyanophyceae later in the summer: Protozoa, rotifera and crustacea may be encountered at any season. However, in the main body of the Wanaque Reservoir, except for the first years of filling, diatoms have prevailed throughout

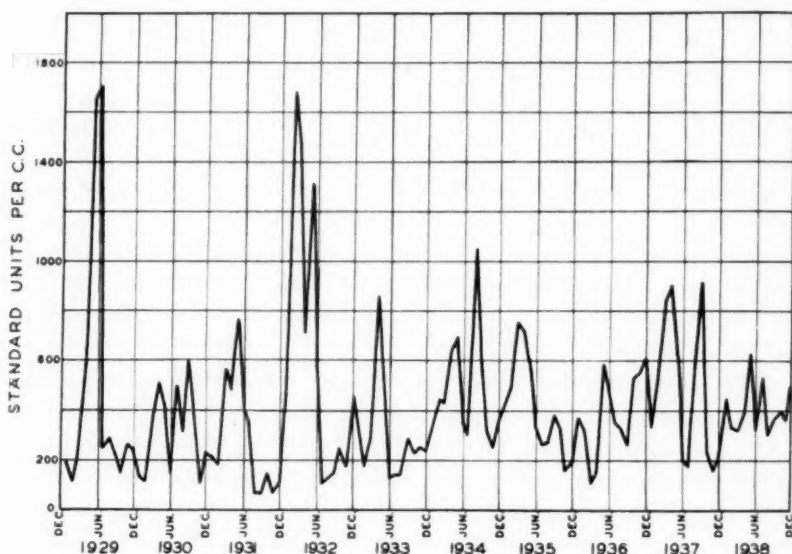


FIG. 7. Micro-organisms—Reservoir Surface

the entire years. Maximum diatom growths—growths large enough in numbers to require a copper sulfate treatment—have been experienced generally in the winter and spring months (fig. 7). A very heavy growth of *Anabaena* (1,836 standard units per c.c.) was noted in the main body of the Wanaque reservoir in 1929 when the reservoir was filling. In the more shallow coves, seasonal growths of chlorophyceae, cyanophyceae and protozoa have been noted. The distribution of diatoms during periods of stratification is greatest above the transition zone (fig. 8). During period of complete vertical circulation there is a uniform distribution of organisms. Following

a copper sulfate treatment, dead algae are commonly found concentrated at the top of the transition zone.

New reservoirs are inclined to produce prolific algae growths during the first years. However this does not infer that old reservoirs are free of heavy algae growths.

Figure 9 indicates the number of microscopic organisms in Lake Erskine previously referred to as an artificial lake development in North Jersey. Copper sulfate treatment is practiced in this lake only in the summer months. Therefore the winter and spring months are peak months. The prevailing organisms at these times have been

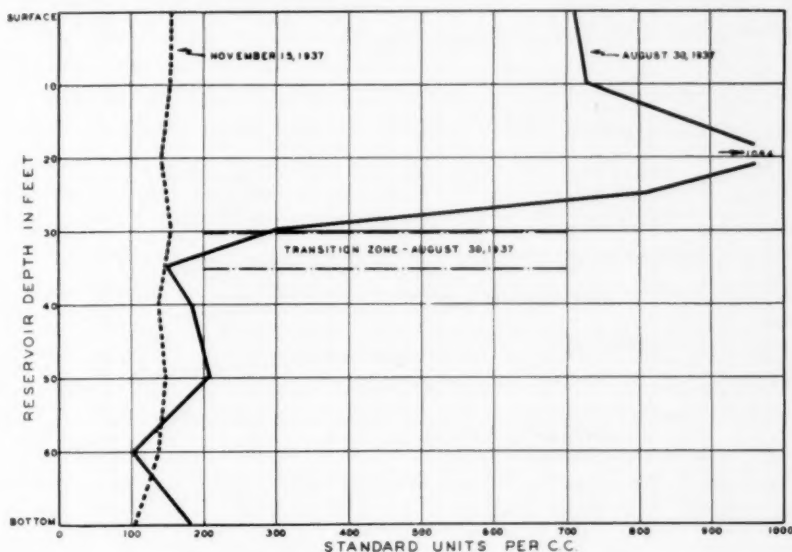


FIG. 8. Micro-organisms—Wanaque Reservoir

diatoms. During the summer season prolific growths of chlorophyceae and cyanophyceae have been prevented by "coppering."

Figure 10 indicates the number of microscopic organisms in Cup-saw Lake. This lake is treated with copper sulfate only during the summer months. This accounts for higher peaks at seasons other than the summer time. This lake, however, characteristically develops *Synura* in the fall season, while from Erskine Lakes one may expect *Dinobryon* at the same season. The writer feels that in certain lakes algae may develop which are characteristic of that lake. There is however much yet to be learned concerning the various factors which control algae development.

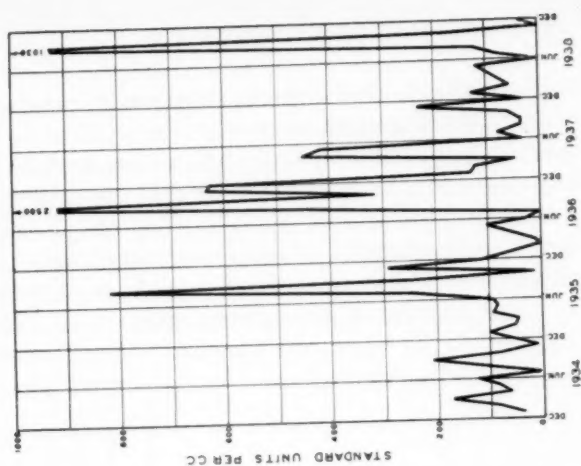


FIG. 11. Micro-organisms—Greenwood Lake

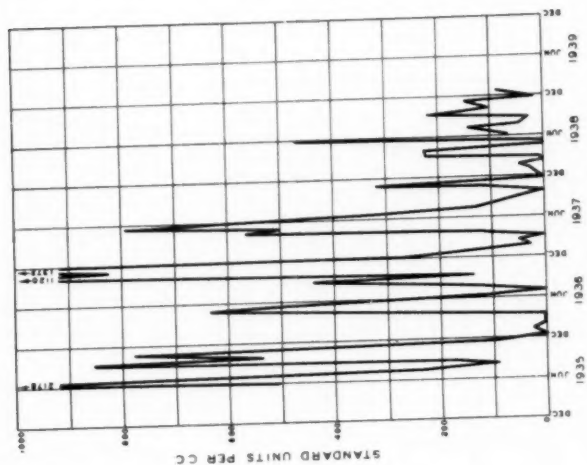


FIG. 10. Micro-organisms—Cupsaw Lake

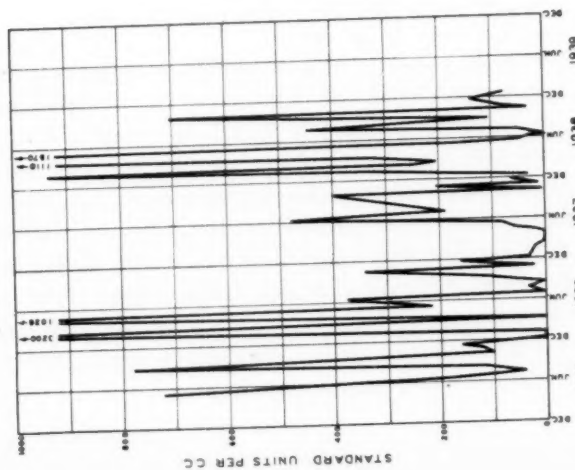


FIG. 9. Micro-organisms—Erskine Lake

Figure 11 indicates the numbers of micro-organisms in Greenwood Lake. This lake is not treated for algae control. This is an example of algae growths in a 103-year-old lake. It is significant to note that excessive growths are less frequent and that the peaks occurred in the late summer and early fall.

The writer has noted that a heavy growth of *Protococcus* lingered throughout a period of years through one season after another in Weequahic Lake, Newark, New Jersey. This organism acquired an apparent immunity to copper sulfate.

In 1928 when the Wanaque reservoir was first filled, a luxuriant growth of *Anabaena* developed in the main body of the reservoir. Since that time there has been no tendency for maximum growths of organisms other than diatoms.

The writer was asked, by the New Jersey State Police, to make an inspection of a certain lake in Northern New Jersey. It was suspected that large quantities of mash from an illegal still were being dumped into the lake. After cutting through 12 in. of ice an inspection of the lake revealed that the water below the ice contained an enormous quantity of organic matter very closely resembling pulp or mash. The ice itself was discolored by the same material. A microscopical examination showed that this organic matter was neither mash nor pulp but decomposing *Aphanizomenon*, an alga of the cyanophyceae group. Cyanophyceae are generally warm weather organisms but this genera is the exception to the rule and has been encountered, as in this case, during the winter months. The cyanophyceae group possesses the faculty of presenting a blue green color effect by transmitted light and a yellow to reddish color in reflected light.

After reviewing the change in characteristics of the Wanaque Reservoir raw water during 10 years of aging, it may be of interest to review pertinent data of the delivered water since 1930. The normal overflow level of the reservoir is 300.3 feet. Flash boards in place raise overflow elevation to 301.67 feet above sea-level. In every year except in 1938, the reservoir dropped below normal overflow level during the month of June and usually continued in its downward trend until November or December, except in 1933, when during August heavy rains raised the elevation above the overflow level. In October of that year the reservoir again went below overflow level and dropped steadily until the month of January when it again began to rise to an overflow level.

The maximum temperatures (between 18° and 20°C. or 64° and

68°F.) have occurred in September when intakes are changed from a 40-foot depth to a mixture of that depth water and some surface water. This method of delivery is practiced to permit the delivery of water which is lower in carbon dioxide and mineral salts.

The maximum manganese values have been reached at the turnover period. These values are more or less controlled by the point of draft from the reservoir to the aqueducts. In each succeeding year there was a longer period of time in which the manganese content averaged less than 0.10 p.p.m. In 1932 and 1933 there were periods of 2 and 2½ months respectively when the manganese content was less than 0.10 p.p.m. In 1934, 1935 and 1936 there were periods of 5, 6 and 7 months respectively in which the delivered water contained less than 0.10 p.p.m. manganese. In 1937, there was a period of only 5 months when the manganese was less than 0.10 and in 1938 there was again a period of 7 months with less than 0.10 p.p.m.

The turbidity dropped from an average of 3 p.p.m. for the years 1930 and 1931 to an average of 2 p.p.m. The maximum turbidities occurred at the fall turnover period.

Color was at a maximum during the fall of 1930 with peaks also at the Fall turnover. High runoff during 1938 caused color values greater than in 1937.

The micro-organisms in the delivered water have been kept below 800 standard units, except in February, 1932, when a growth of *Asterionella* reached nearly 2,000 standard units.

Generally speaking the amorphous matter has been lower than the number of micro-organisms except during the fall season of the first two years and the fall of 1938.

### Conclusions

A reservoir may not be considered as having come fully of age until the impounded water therein has ceased to improve. This period of aging varies with different characteristics. For example the apparent color of the Wanaque Reservoir, the basin of which was cleared of trees to the ground level and where there was complete removal of trees for a distance of 5 ft. above and below normal flow line level, reached a minimum in the bottom samples seven years after filling. Following this period the color in the reservoir bottom was influenced contrariwise by runoff and drawdown of the reservoir. It therefore follows that the period of color aging is dependent among other things on the storage ratio of the reservoir. It would seem

that when the storage ratio is approximately 1.0 the practical period of color aging may be of 7 years duration. The color aging time of an unused reservoir of low storage ratio may be as low as four years in a reservoir from which top soil has not been stripped. When a reservoir flooded area has been stripped and swamp areas removed, maximum color reduction may take place in a shorter time.

From the standpoint of mineral salts, such as iron and manganese, in the bottom water there is a vast improvement during the first three years. The manganese improvement apparently is slower than the iron improvement.

The carbon dioxide in the Wanaque Reservoir appears to have reached a nearly stable minimum in the bottom water in the sixth year.

The dissolved oxygen content in the Wanaque Reservoir was zero in the bottom water during the fall seasons of each of the first three years. After the first three years dissolved oxygen was always present even in the bottom water. The length of time of minimum dissolved oxygen decreased and the quantity of dissolved oxygen in the bottom water increased gradually following the third year.

Hydrogen sulfide was present during the summer and fall seasons of the first three years in the bottom water and extended upward through the transition zone to the surface during the first year. Following the first year there was no hydrogen sulfide at the surface. There has been no hydrogen sulfide present even in the bottom water since 1932.

The numbers of micro-organisms may be excessive at times even in old reservoirs. During the first year of filling, if filling takes place during a growing season, a large reservoir may have excessive growths of Chlorophyceae and Cyanophyceae and in succeeding years that reservoir may not again experience troublesome green or blue green algae conditions except in the more shallow coves.

Foresight on the part of reservoir constructing agencies in the collection of analytical data on streams which at some future date may be impounded for water supplies is recommended. These data should include sanitary chemical analyses as frequently as possible. The North Jersey District Water Supply Commission has collected and analysed data on the more important potential water supply streams throughout the North Jersey District from 1923 to July, 1932 at quarterly intervals. Such data are available and indispensable when the construction of new reservoirs is considered.

The writer wishes to acknowledge with deep appreciation the coöperation of Chrystal Brown, Superintendent of the North Jersey District Water Supply Commission, in the preparation of this paper.

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**Discussion by Robert Spurr Weston.\*** Mr. Purcell's paper is an important contribution and compares with the studies of the Nepaug Reservoir presented by Caleb Mills Saville (1) and with those of some stripped reservoirs in Massachusetts by Karl R. Kennison (6).

It is extremely gratifying to those who planned the sampling schedule during the construction period of the Wanaque Reservoir to find that Mr. Purcell has not only continued but amplified it. As he states, the North Jersey District Water Supply Commission with the coöperation of the New Jersey Department of Health has accumulated and is accumulating results of analyses of the surface waters of New Jersey which will be of inestimable value to the water works planners of the future.

Perhaps the best way to contribute to the discussion of Mr. Purcell's observations and illuminating diagrams is to review the mechanism of purification by storage, particularly decolorization, clarification, and reduction in growths of microscopic organisms.

Decolorization by storage is an old subject. It was mentioned in Prof. William Ripley Nichols' book on water supply which was published in 1883. It was studied by Desmond Fitzgerald, Chief Engineer of the Boston Water Board, and his assistant William E. Foss, in the years 1891-94, and in great detail by Frederick P. Stearns (2), Chief Engineer of the Massachusetts Board of Health. It was treated at length at an informal discussion before the American Society of Civil Engineers in 1901 (3). It was the subject of an impor-

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\* Consulting Sanitary Engineer, Weston & Sampson, Boston, Mass.

tant paper by Ralph H. Stearns (4) in 1915. Whipple, Hale and others have referred to the decolorization of the Croton water in various communications, and decolorization was carefully considered by C. M. Saville (5) with the late Frederick P. Stearns and the writer in connection with the Hartford water supply. It has been given consideration by designers and operators of water works, including Hazen, Fuller, Lochridge, Stanley, Eddy, Pirnie, Winsor and others.

At one time stripping was thought advisable in spite of its cost. It still is, for quick results. The amount of \$2,536,612 was spent to strip the Wachusett Reservoir and it aged quickly. Before long, however, it was realized that in a reservoir a few years old the character of the deposits from the water rather than the bottom which they cover was what ultimately affected quality by contact. Modern reservoirs are cleared, but, excepting between the low and high water marks, are not stripped, and filtration is employed where decolorization is inadequate.

Two entirely different waters—surface and ground—enter a reservoir. The former contains dissolved and colloidal vegetable matter most of which is derived from the green coloring matter in plants (chlorophyl) which contains iron. Drained from above peaty deposits in swamps, the water may have a color as high as 1,000 like that from Dismal Swamp in Virginia; drawn from a granitic hillside in New Hampshire, it may have a color of less than 10.

Most of the ground water will enter a reservoir with the surface water through the feeders, but if its dam stops the valley a large amount will flow into the reservoir through its sides and bottom and down the valley-ways of the feeders. This ground water will be clear, oxygen-free, and contain carbonic acid. It will contain dissolved minerals, including iron and manganese, in large or small amounts.

The color of the surface water is not all in true solution. The whaling ships used to prefer to take Dismal Swamp water on their voyages because it "worked" and left a limpid, agreeable water above a stable ooze, or, as a physical chemist would say, the protective colloidal organic matter decomposed in the presence of oxygen while agitated by the movement of the vessel, thus permitting the iron to form flocs with the altered organic matter and clarify the water, and also cause the organic matter to decompose without the production of bad taste.

In the earlier discussion referred to by Mr. Purcell, it was stated

that decolorization is due to the bleaching action of light, but there was no light in the whalers' water butts and we know now that the bleaching action of sunlight in a reservoir is largely confined to the surface layer. Circulation occurs, however, and decolorization by light is, therefore, an important factor as the work of Kennison (6) shows.

During the summer period of stagnation, the depth of circulation above the stagnant stratum or the thermocline is due to the temperature of the air. The more the surface layer is cooled, as during the nights, the deeper it will go before its temperature is equalized and movement stopped. Finally, the turnover is complete, conditions are uniform and oxidation and flocculation occur, followed by the settling out of some of the coagulated organic matter, iron and manganese, which are the chief components of what is called color.

During the period of stagnation, there has been an accumulation of coloring matter dissolved from the soil and also organic coloring matter deposited from the water itself, like that deposited on the bottom of a bottle of colored water exposed to the light or in the whalers' water butts.

The bacteria decomposing the organic matter, as Mr. Purcell's data show, may exhaust the dissolved oxygen and even rob the nitrates and sulfates of their oxygen. Some bottom waters stink with hydrogen sulfide and are black with iron sulfide.

Because the decomposition of the organic matter produces carbon dioxide and because some carbon dioxide enters with the ground water, the water is acid, and iron and manganese are dissolved from the bottom or the bottom ooze. Both of these elements may be in the form of carbonates and, when hydrogen sulfide is formed, of sulfides also.

When brought to the surface with the turnover and in contact with oxygen, the salts of iron and manganese are oxidized to the ferric and manganic hydroxides, the carbon dioxide and sulfur being set free. The hydroxides now await a condition favorable to coagulation.

After decomposition of part of the organic matter has taken place, the iron and manganese hydroxides are not prevented from passing from a finely divided colloidal form to the floc form, when they subside and clarify.

As Dr. Frank E. Hale (7) has shown, manganic hydroxide coagulates more slowly and forms finer flocs than does iron hydroxide;

therefore it is more likely to be delivered to consumers during the critical periods in the year, like that following the turnover.

More mathematics is required to answer Mr. Purcell's question regarding the advantage gained by draining off stagnant bottom water. It is a common practice however. It sometimes brings down the top water and causes an unnatural turnover (7).

The best single index of controlling factors in a reservoir is the storage ratio, and this is the resultant of runoff and storage. Mr. Purcell has given examples of the effects of the new Erskine and Cupsaw Reservoirs in which storage is nil; of the effect of the century-old Greenwood Lake with a storage ratio of 1.24 and of the new Wanaque Reservoir with a storage ratio of 0.70. In another place

TABLE 1  
*Reduction of Color in Reservoirs*

NO. OF DAYS STORAGE	% REDUCTION IN COLOR
50	19
100	27
150	33
200	38
250	42
300	46
350	50
400	53
450	56
500	60

(8), the relation between the storage ratio and the numbers of microscopic organisms in New Jersey reservoirs has been shown. In these the numbers were large when the water was stored for less than eight months. All know the effects of the Great Lakes in this regard.

About fifteen years ago the writer averaged all of the available data regarding reduction of color in reservoirs, in the form of a curve. Part of the curve showed the following values as given in table 1.

Some of the reservoirs, like the Wachusett in which the period of storage was over 600 days, were stripped; others were not.

Using the data in this table, it may be estimated that the reduction in color by storage in the Wanaque Reservoir will be about 25 per cent. Mr. Purcell records a reduction of 21 per cent so far and the reservoir is not yet aged.

As the years go by, the water of a reservoir becomes less and less affected by the bottom soil and its own deposits become more controlling. More and more iron accumulates but whether bog iron ore will be formed beneath the bottom deposits as is formed in swamps must be answered by the geologists of the next era. Mr. Purcell will have left them a good record of beginnings.

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## ABSTRACTS OF WATER WORKS LITERATURE

**Key.** 30: 402 (Mar. '38) indicates volume 30, page 402, issue dated March 1938. If the publication is paged by issues, 30: 3: 402 (Mar. '38) indicates volume 30, number 3, page 402. Material inclosed in starred brackets, ★[ ], is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.*—*Bulletin of Hygiene (British)*; *C. A.*—*Chemical Abstracts*; *P. H. E. A.*—*Public Health Engineering Abstracts*; *W. P. R.*—*Water Pollution Research (British)*.

### HEALTH AND HYGIENE

**The Public Health Service Leaves the Treasury Department.** BROCK C. HAMPTON. U. S. Pub. Hlth. Rpts. 54: 1133 (Jun. 30, '39). The Public Health Service began as the Marine Hospital Service July 16, 1798, when it was created for the care of sick and disabled American merchant seamen, more often incapacitated away from home, at sea or in foreign ports. For example, in 1801, Evan Jones informed the Sec. of State that in New Orleans, then under Spanish rule, that many seamen and river boatmen die there yearly from lack of hospitalization, and suggested a small tax on each man and vessel to provide proper facilities. Va. and Mass. were first in America to provide hospital and medical care for seamen away from home. The first U. S. hospital actually provided for seamen under the Federal Act of 1798, establishing the Marine Hospital Service, was the Va. Hospital at Washington in 1801, although the first treatment under the Act was given in Boston, 1799. Later, marine hospitals were established at various Atlantic, Pacific, important river and Great Lake ports. The administrative organization became known as the Marine Hospital Service. Growth and evolution of public health functions from this beginning was along natural and logical lines. The medical officers were often the first to diagnose cholera, yellow fever and small-pox then being imported. Epidemic control and quarantine enforcement, normally state functions, were gradually assumed by the M.H.S. In '78, the Service was given broad powers for cooperating with state and local health authorities in control of disease. In '90, it was authorized to aid in preventing introduction of cholera, yellow fever, small-pox and plague. In '93, its quarantine powers were made to include all infectious and contagious diseases. In '89, the Service officers were commissioned in grades similar to those of the medical department of the Army. In '17, Executive order provided that the P.H.S. shall constitute a part of the military forces in time of threatened or actual war. From '93, additional health functions were continually imposed. As a result, in '02, the name was changed to U. S. Public Health and Marine Hospital Service,

In '12, it became the U. S. Public Health Service alone in accord with its enlarged public functions. Besides the important part the Service played in combating and controlling yellow fever, an outstanding achievement was the eradication of bubonic plague in San Francisco against organized opposition. Investigative research was officially authorized in '01 with subsequent building of the Hygienic Laboratory in Washington, D. C. In '12, research functions were extended to study and investigation of all human diseases, the conditions influencing their propagation and spread, including sanitation and sewage, and the pollution directly or indirectly of navigable streams and lakes. In '02, supervisory power and authority were given to regulate sale of viruses, serums, toxins, etc., shipped interstate. In '17, the Service was required to physically and mentally examine all arriving aliens and intending immigrants. An important advance occurred in '18 with the creation of the Division of Venereal Diseases, authorizing cooperation with state departments of health for control within the state and prevention of spread interstate. These activities were vastly extended in '38, providing annual grants-in-aid to states and local districts to establish and maintain adequate preventive, treatment and control measures in relation to venereal diseases. For 3 years, '19-'22, the Service bore the heavy emergency responsibility of providing hospital care and treatment for sick and disabled veterans of the World War. The most important recent legislation affecting the P.H.S. and promoting public health are the Social Security Act of '35, the National Cancer Act of '37 and the Venereal Diseases Act of '38. These reflect the changing concepts in public health and changing emphasis on various public health activities. After a life of nearly a century and a half under the administrative jurisdiction of the Treasury Dept. the Service left it 7-1-39. It began there for fiscal reasons but subsequent growth into an independently functioning and scientific organization made this relationship anomalous. On the whole, however, its progress and successful functioning has been due in large part to the sympathetic understanding, ability and forward-looking attitude of the Treasury heads. Transfer has been made to the Federal Security Agency where Federal health activities may be better carried out. While this transfer carries certain sentimental regrets it marks a new era for public health. Article is illustrated.—*Ralph E. Noble.*

**Detection of Two Typhoid Carriers.** J. WYLLIE. Canadian Pub. Health J. 30: 161 ('39). First case was a woman who had had typhoid 8 years previously and was only found to be a carrier on investigation after her son had become infected. By Craigie's phage tests the identity of the strains was established. It was further shown that she had probably previously infected her husband and another son and, from their agglutinin titres, was possibly responsible for subclinical infection of three other members of the family. The other case, which occurred in an institution, was a woman whose agglutinin titre rose again 5 mos. after her original infection and was maintained at a high level for 8 mos. This probably indicated the multiplication of bacilli in the gall bladder for she was found to be a carrier, probably responsible for another case in the institution.—*B. H.*

**Typhoid Endemicity; the Carrier Problem.** K. HOESCH. Offentl. Gesundheits-Dienst. 4: 785 ('39). This paper is of interest on administrative rather than

general epidemiological grounds. It concerns a small group of persons resident in a hamlet not far from a town and used as a pleasure resort (names not given, but probably located in the Rhineland). In '26 nine cases of enteric fever occurred. It seems probable that the original source of infection was drinking water perhaps polluted from the town. Among the patients were the wife of a farmer and her son. Ten years later enteric fever broke out again in the house of the farmer, three lodgers were attacked and one died. It was now established that the wife was a bacillus carrier and her husband became involved in complicated lawsuits. The wife was exhaustively examined in hospital and operation advised. The family by now was in distressed circumstances (they lived by selling milk) and could not pay the cost; although this was apparently reduced, the patient did not submit to operation; shortly after, her daughter went down with enteric fever and died as did the nurse who attended her. This completed the ruin of the family. The author points various morals, e. g., stricter control of carriers and free provision of adequate treatment.—*B. H.*

**Typhoid Fever.** ANON. Bacteriological Investigations of the U. S. Pub. Hlth. Serv. Suppl. No. 141, p. 64. Typhoid fever was isolated in the Hygienic Laboratory by Kinyoun as early as 1889. In 1895 the Service gathered statistics representing a total population of 5,000,000 people and found a death rate from typhoid fever of about 60 per 100,000. In connection with typhoid surveys many water supplies were investigated, and during the several succeeding years there was much public agitation for a filtered water supply for the District of Columbia which finally culminated in the installation of a filter plant. To the consternation of authorities there was an actual increase in typhoid following the setting up of the new filtration plant and the investigations of Lumsden, Anderson, Rosenau, and Kastle resulted in publications which represent one of the most valuable contributions America has made to epidemiology. During these studies a number of younger officers were trained and prepared for valuable service in connection with many other diseases. In 1911 the Service campaign against typhoid was well under way. Lumsden was finding that the way to rural control lies in the establishment of county and regional health organizations on a full time basis. In the registration area of the U. S. the typhoid death rate declined from 35.9 per hundred thousand in 1900 to 5.5 per hundred thousand in '38. This decline was due almost entirely to recognition of proper water purification methods and proper milk control.—*P. H. E. A.*

**Infectious Diseases (except Abdominal Typhus) in Water, Especially Drinking Water.** H. BRUNS. *Vom Wasser* 13: 14 ('38). Pathogenic bacteria will develop in water under 20°, especially water containing organic matter. Some natural factors in their destruction are dilution, sedimentation and sun light. Some part is also played by lower plants and animal life and certain bacteriophages. Activated aeration causes considerable destruction. However, many diseases come from drinking waters into which human excretions, sputum, etc. have entered. Numerous epidemics of a serious nature are noted, with some statistical data. Author claims to know of no worse infection than acute gastro-enteritis, gives some data and comparisons of this and other

diseases. Drinking waters of all kinds should be treated by disinfection with chlorine-lime treatment.—*Willem Rudolfs*.

**Investigations of the Sewage in the Neighborhood of Riesa and Breslau for the Presence of Bacteria of the Paratyphoid-enteritis Group.** K. SAGER. Inaugural dissertation, Friedrich-Wilhelms Univ., Berlin, '36. Reference made to 34 papers, of which brief abstracts are given, dealing with the source of outbreaks of typhoid, paratyphoid, and gastro-enteritis. 172 samples of water were taken from sources polluted by town sewage or by birds and animals, and the characteristics of the bacteria isolated from the samples were investigated. Even in samples taken from water in places where animals suffering from enteritis had been present for some time, no bacteria of the paratyphoid-enteritis group were found. The investigation lends no support to the supposition that water plays an important rôle as a source or carrier of paratyphoid-enteritis diseases.—*W. P. R.*

**Shiga Dysentery.** ANON. Ill. Hlth. Messenger 10: 17: 1 (Sep. 1, '38). Shiga dysentery is caused by one of the several strains of *Bacillus dysenteriae* which is a very virulent strain first discovered by the Japanese scientist, Shiga. Of the 50 to 100 deaths attributed to bacillary dysentery, and of the 800 to 1,200 from diarrhea and enteritis, it is conservative to estimate that Shiga dysentery is responsible for 50 to 100 deaths annually in Illinois. The germs of bacillary dysentery are difficult to identify but vary greatly in their virulence causing both fatal and mild illnesses. It is spread in the same ways as typhoid.—*P. H. E. A.*

**Copper and Health.** G. PY. Cuivre et laiton (Fr.) 12: 127 ('39). Comprehensive treatise on sanitary importance of copper used in cooking, preserving and plumbing. Normal drinking water contains up to 0.5 mg./liter, while more aggressive waters have up to 5 mg., which entering into foodstuffs can reach 1 mg. per day normally without endangering health. Total copper that can be absorbed by man is about 20 mg. per day. Copper acts differently, however, when absorbed by empty stomach or after meal; in latter case, it is practically entirely absorbed by vegetable pulp and eliminated. Therapeutic value of copper is discussed.—*C. A.*

**Lead in Food.** G. W. MONIER-WILLIAMS. Ministry of Health Reports on Pub. Health & Med. Subjects. 88: iv. +51 ('38). The various ways food may be contaminated with lead are set out generally and some are discussed in detail. A useful table shows the amounts of lead, in p.p.m., in various foods, based on analyses made in the Food Laboratory of the Ministry of Health. In nearly all cases the findings show less than 4 p.p.m. but in some preparations, mostly food colors, figures as high as 85 to 337 are recorded. A detailed account is given of the absorption and excretion of lead in the body. Owing to the nearly universal presence of traces of lead practically everyone excreted small quantities of lead through the kidneys, usually about 0.05 mg. daily. Lead is readily absorbed from the intestine. Much of absorbed lead is excreted by the liver into the small intestine and eliminated by the feces.

No appreciable amounts are stored in the liver or other organs but a good deal is stored in the skeleton. The bones of a normal individual may contain from 50 to 100 p.p.m. of lead. With a cumulative poison like lead the important point is not whether any possible permissible limits are exceeded but what is the relationship to the total intake of lead. The lowest limit of safety for lead is unknown so that it is desirable to reduce the lead in the food to the lowest amount possible. Specific standards of permissible limits are undesirable and a general reduction of lead in foods by administrative action is better policy. This valuable report also includes a detailed review of methods for the determination of lead in food.—*B. H.*

**Water-borne Outbreaks of Typhoid and Diarrhea.** ANON. Calif. Dept. of Pub. Hlth. Weekly Bull. p. 98 (Jul. 16, '38). Committee on Water Supply of A. P. H. A., in its report on water-borne outbreaks of diseases in U. S. and Canada for '30-'36, reports 170 such outbreaks—3,180 cases of typhoid fever and 17,458 cases of diarrhea, a total of 22,095 individuals having been affected. During this period, nine outbreaks occurred in Calif. These involved 171 cases of typhoid and 175 of diarrhea, 348 individuals having been affected. Of these 171 cases of typhoid in Calif., 105 occurred in the Sacramento Delta Region, 61.4% of the total reported during the seven years. These river outbreaks were multiple, having occurred in '30 and again in '31. None of the nine outbreaks reported in Calif. were due to failure upon the part of any municipality or public service corporation to provide potable water supplies for consumers. Most were due to following causes: (1) Surface pollution of shallow wells. (2) Contamination of surface springs. (3) Cross-connections with polluted water supplies. (4) Inadequate treatment. A very large proportion of outbreaks was reported as "cause undetermined." It is believed by the committee that if these outbreaks were distributed on the basis of probable points of contamination, the cross-connection classification would rank next to surface pollution of shallow wells as a cause of these outbreaks.—*P. H. E. A.*

**Typhoid Outbreak Arouses Suspicion.** Ill. Health Messenger. 10: 19: 111 (Oct. 1, '38). Late in July a job of repair and improvement on Manheim Road through Bellewood, a community in Cook County, was underway. Shortly thereafter an outbreak of typhoid fever involving 5 cases occurred in the vicinity. A cause and effect relation between the two events is suggested by evidence revealed through an investigation conducted by the State Dept. of Public Health. The sewer main in that particular vicinity, it appears, lies above the water main. During the road improvement work, which involved the moving of a hydrant, it appears that both the water and the sewer mains were cut and that both were open simultaneously for a period. Although, according to reports, pumps were used constantly to carry off the sewage which accumulated in the pit where the men were at work on the mains, it may be that rain water, contaminated with sewage at the pit, found its way into the water main, carrying pollution into the homes served by the water mains in question. The time element lends weight to this hypothesis. The mains are said to have been open on July 26 and 27. Date of onset of illness in the two

typhoid fever patients was given as July 30 and the other three later up to August 8. These onset dates indicate that infection might well have taken place at about the time when the mains were open. Evidence that water in the mains was polluted at the time, and because of the work on the mains incidental to the road improvement job, is not absolutely conclusive. It is suggestive. In any event the circumstances betray possibilities and emphasize afresh the importance of expert precautions when work of any kind relates closely to drinking water supplies. *Sewer Contaminates Well.* Another kindred situation came to light at Danvers in McLean County in August. An investigation as to the source of a case of typhoid fever in a local housewife revealed that at least six cases, one fatal, had occurred in the vicinity within a year. Further study showed that a sewer line ran across the street diagonally very close to each of two wells, one on either side of the street. A padlock had been placed on one of the wells some time previously because the water was regarded as unfit to drink. Typhoid had developed in a family that used the other well. Another family later moved into the house served by this well and either didn't know of the previous experience or were heedless. The housewife of this family came down with typhoid. Now both wells are padlocked. Available evidence indicates that the wells are subject to pollution from the sewer pipe. These two histories show clearly the importance of constructing and maintaining sewer and water systems in strict compliance with established sanitary principles and the danger that always lurks behind any violation of these principles.—P. H. E. A.

#### CROSS-CONNECTIONS—PLUMBING HAZARDS

**Ending Cross-Connections in San Francisco.** N. A. ECKART. W. W. Eng. 92: 1103 (Aug. 30, '39). Following joint survey of City Water and Health Dept's., from Jun. 1, '38, to Jan. 1, '39, all San Francisco, Calif., hotels and rooming houses may be considered safe from possibility of pollution of water supply through cross-connections. In '33-'34, previous to bringing of Hetch Hetchy water sources to city, similar survey disclosed some 231 active cross-connections with other sources of supply which were then either eliminated or corrected. Present survey dealt with cross-connections within the buildings themselves involving water supply and sewage system or other auxiliary facilities. In all 651 hotels inspected, of which 82 or 12.6% maintained one or more serious hazards within building piping. 27 hotels contained 28 cases of direct sewage-water connections (4.2% of total defects found); in 11 hotels 14 cases (1.7%) of equipment with submerged inlets were reported; 4.5% numbering 1359 examples of fixtures with submerged inlets were reported as existing in 29 hotels while in 55 hotels there were found 59 defects of primary lines to pump ejectors. Hazardous conditions were promptly cleaned up, all hotel owners providing excellent co-operation in carrying out work.—Martin E. Flentje.

**Double Check-Valve Maintenance and Tests in Connecticut.** FREDERICK O. A. ALMQUIST. J.N.E.W.W.A. 52: 379 (Dec. '38). Brings up to date double check-valve tests from previous '27 report, includes comments and procedure for maintenance of check-valve installations. Conn. regulations allow no

additional cross-connections. On connections existing Dec. 31, '26 two gate valves with indicator posts, 2 check valves of approved design with drip cock and gages for testing all located in water-tight vault may be permitted. Full dep't. regulations given. Approved check valves include so-called Factory Mutual (F.M.) and the bronze Rice or Underwood valve. At present time there are 151 active installations in Conn. Check valves should be spaced from 3' to 5' apart to prevent long object in pipe holding both valves open at same time. Procedure for checking given, Health Dep't. does own work, at first check was made every second month, after several yrs. increased to every 3 mo., now made every 4 mo. Annual cleaning of valves found desirable. Of 6823 tests from Jan. 1, '27 to June 30, '38 there were 122 leaks in outside valves 17 in inside valves, and 7 in both valves; since '29 only one installation found leaking in both valves. Tests tabulated by yrs. in considerable detail. Better results obtained by all-bronze valves than for iron-body valves. 12 yr. test results show great degree of reliance can be placed on these valves for safe-guarding public water supplies, provided valves are of approved type, preferably all-bronze, and properly maintained. Periodic inspections unquestionably necessary. Req'd. maintenance should include yearly opening of check-valves for thorough cleaning and overhauling. *Discussion:* New Bedford 27 yr. experience with double check-valves on fire-supply services; 5648 tests covering 83 sets of valves from 4" to 10" in size. During 5 yr. period, '33 to '37 incl., in the 1694 inspections made there was no case where both check-valves leaked and only 1 case where one check leaked. Shows value of quarterly inspections and annual overhaul. Fifteen sets of iron-body double check-valves with aggregate life of 640 valve-years never found leaking in 1015 inspections. Two all-bronze sets with 28 yr. aggregate record inspected 63 times and never found leaking. Insurance companies recommend testing for tightness by owner once each mo., and complete annual inspection.—*Martin E. Flentje.*

**Public Water Supply Contamination By Way of Cross-Connections in Sandusky, Ohio.** O. F. SCHOEPFLE. Ann. Rept. Ohio Conf. Water Purif. 18: 103 ('38). Complaints of turbid water from distribution system led to discovery of industrial supply connected to public system through swing check valve which was broken, allowing valve to remain in open position. Pressure of full elev. tank on industrial system during shutdown of plant exceeded that in public system and allowed raw water to enter latter. All persons in affected area were advised to submit to inoculation, which was done free of charge. No sickness resulted. This cross-connection and several others disclosed by comprehensive survey were eliminated. There should be greater cooperation between fire insurance and health authorities. Former were aware of existence of the cross-connection in violation of state code.—*R. E. Thompson.*

**An Outbreak of Typhoid Fever Caused by Cross-Connection with Plumbing Fixtures.** B. V. BEVIER. Ann. Rept. Ohio Conf. Water Purif. 18: 102 ('38). Two small outbreaks of typhoid in Middletown, Ohio, were traced to siphonage of sewage from plugged sewer through riser drains of yard hydrants. Yard hydrants and frost-proof toilets on certain street were served by 4" main paral-

leled closely by 6" sewer on flat slope to which toilets and waste water drains of hydrants were connected. Sewer had been plugged and water main had been emptied for repairs on several occasions. Epidemics might easily have been more widespread.—*R. E. Thompson.*

**Sanitary Installations for Households.** ROBERTO J. PERAZZO. *Bol. Obras Sanitarias Nacion* (Buenos Aires) **2**: 357 (Oct. '38). Brief introduction of the subject of cross-connections is followed by detailed and well-illustrated description of the experimental work being done, for the better understanding of the problem, at the Universities of Iowa, Wisconsin, and Minnesota, and at the National Bureau of Standards.—*J. M. Sanchis.*

**Plumbing-Borne Diseases.** R. ST. J. MACDONALD. *Can. Pub. Health J.* **30**: 198 (Apr. '39). During past 20 yrs. there has been tendency on part of many public health authorities to relegate certain phases of sanitation, notably plumbing and plumbing equipment, to positions of minor importance. Disease outbreaks in recent yrs. have emphasized, however, that sanitation is still fundamental basis of any structure for promotion of health and prevention of disease. Mechanism of transmission of disease by plumbing is reviewed and reference made to specific outbreaks attributed thereto. In case of direct cross-connections, essential need is to require that only safe water supply be permitted in buildings. Danger of indirect cross-connections may be avoided by providing efficient vacuum breakers or flush valves several inches above overflow line of fixtures. Cause of vacuum in plumbing system should be investigated and eliminated. Safe and unsafe types of fixtures are illustrated. List of safety measures which should be observed is appended.—*R. E. Thompson.*

**Vacuum Frequency—The Measure of Hazard in Cross-Connections.** ARTHUR B. CRONKRIGHT AND ARTHUR P. MILLER. *Eng. News-Rec.* **123**: 88 (Jul. 20, '39). To permit better understanding of results of plumbing survey of federal bldgs. in New York and Detroit, study was made of 6 office bldgs. to determine frequency with which vacuum or less-than-atmospheric pressure occurred in water supply piping. Method first adopted was to install recording gage in that part of system considered, from visual inspection, as most likely to evidence vacuum. Owing to length of time necessary in some instances for vacuum to be recorded, revised procedure was adopted based on probability theory. Pressure chart records were obtained as before and statistical array made by selecting lowest pressures recorded in each consecutive 15-min. interval and arranging them in order of magnitude. Frequency of occurrence of each interval of pressure was determined and recorded and from this per cent of pressures recorded less than each stated pressure was calculated and plotted on both arithmetic and logarithmic probability paper against stated pressures. Extension of lower ends of resulting curves to intersect line of 1 lb. pressure permitted approximation to be made of frequency with which that pressure could be expected to occur. The 1 lb. pressure was used in preference to zero in order to permit reading corresponding values from logarithmic plats. Frequency thus determined was considered as vacuum frequency of bldg.

Using this data, relative hazard represented by no. of defective plumbing fixtures or installations may be evaluated and dangers inherent in plumbing systems of 2 bldgs. may be compared. Vacuum frequency (%) in 5 of bldgs. varied from negligible to 0.24 and in 6th was 25. Obviously the same no. of potentially hazardous cross-connections would be of vastly greater significance in latter than in other 5. Condemnation of plumbing system or installation in bldg. requires more than simple inspection of various connections and fixtures.—*R. E. Thompson.* (See also abstract J.A.W.W.A. 30: 719 (Apr. '38)).

**A "Pumpless" Air Compressor.** W. F. SCHAPHORST. Ind. Eng. Chem.—News Ed. 16: 672 (Dec. 20, '38). Compressed air in small quantities can be provided by closing valve at top of an ordinary water tank and allowing water from city main to enter at bottom until pressure equilibrium is reached. Trapped air can be withdrawn as needed by opening upper valve, admitting more water from below. When full, tank is drained and process repeated.—*Selma Gottlieb.* (If the lower valve is left open or if it leaks, and the water pressure falls, back flow of the water would occur.—*Ed.*)

#### WELLS AND GROUND WATER

**New Formulas for the Calculation of the Yield of Wells.** A. VIBERT. Compt. rend. acad. sciences (Fr.) 208: 454 (Feb. 6, '39); also more complete *Génie civil* (Fr.) 114: 216 (Mar. 11, '39). In this theory it is assumed that the surfaces of equal potential are formed by concentric ellipses with constant eccentricity and having the small axis at the base of the water-bearing strata. The formula developed is:

$$q = \frac{2\pi R \epsilon \beta^2 H^2 \left[ 1 - \left( \frac{y}{H} \right)^{\beta^2} \right]}{R - x}$$

Where  $R$  is the radius of influence,  $H$  the undisturbed static head or depth of water,  $y$  the depth of water at the distance  $x$  from the center of the well,  $\epsilon$  the coefficient of permeability and  $\beta = \frac{b}{a} < 1$  of the ellipse. Test shows that this gives a draw down curve corresponding to the actual draw down curve. This curve lies much higher than that found by the theory of Dupuit.—*Max Suter.*

**The Flow of a Liquid in a Resistant Media.** JEAN FOURTANÉ. *Génie civil* (Fr.) 114: 277 (Apr. 1, '39). Using Euler's and Bernoulli's equations and a law of resistance of the type  $KW^m$ , where  $K$  is a constant,  $W$  the velocity vector opposing the influence of gravity and  $m$  an exponent of a value between 1 and 2, formulas are developed that give the pressure loss as a function of the radius of curvature of the flow lines. As the full integration is difficult, circles are used for the shapes of the flow line, all having their centers in the plane of the circle of influence. It is shown that in such a case,  $W$  is a lineal function of the reciprocal of the drop in head and that under Darcy's law it is necessary that  $M = 1$ .—*Max Suter.*

**The Porosity of Quartz Sands in Netherland Determined by Geo-hydrological Examinations.** B. A. VAN NES. *Water (Neth.)* **23**: 139 (Jul. 28, '39). For the solution of practically all questions in the geo-hydrologic field a knowledge of soil porosity is necessary. The porosity is expressed as coefficient  $k$ , indicating the velocity of a definite quantity of water passing perpendicularly in a specific time at a given pressure loss through a unit surface. This porosity coefficient can be determined in the laboratory, but to determine the capacity of a well or wellfield pumping tests are preferred. A pumping test can be made by construction of a pump pit and two observation pits placed at different distances from the pump pit. The changes in water levels are recorded and the porosity calculated. Assume: waterbearing sand covered and underlaid with impervious clay layers; the infiltration is directly proportional to the lowering of potential of ground water in the sand, expressed as

$$z = \frac{Q}{2\pi mk} K_0(ax) \dots \dots \dots (1)$$

$m$  = thickness of sand layer in meters,

$k$  = porosity coefficient in the sand in m./hr.,

$K_0(ax)$  = Bessel function to the order of 0 with  $a$  as a constant. The value of  $a$  is given by:

$$a = \sqrt{\frac{k'}{mkD}} \dots \dots \dots (2)$$

$D$  = thickness of cover layer below the phreatic surface in meters,

$k'$  = porosity coefficient in this layer in m./hr.

With aid of (1) and (2) and the values obtained during pumping,  $m$ ,  $K$ , and  $k'$  are calculated. Formula (1) is valid as long as precipitation or surface water is sufficient to keep constant the phreatic surface in the cover layer. In practice more than two observation pits are needed. Formula (1) holds for pits of small diameter. The general formula for the yield  $Q$  (in m.<sup>3</sup>/hr.) of a circular pit by pumping  $z_r$  (in m.) is:

$$Q = 2\pi mkz_r a \frac{K_1(ar)}{K_0(ar)} \dots \dots \dots (3)$$

$r$  = radius of pit in meters,

$K_1(ar)$  = a Bessel function to the order of 1. If no feeding takes place by surface water and the precipitation is insufficient for replacement, the phreatic surface in the cover layer will be lowered until the pressure difference between the phreatic surface and pressure is just sufficient to allow infiltration of the useful precipitation. Tests and operation results show that a sand layer of 200 m. thickness had an average porosity coefficient of 1 to 1.5 m./hr.—*Willem Rudolfs*.

**Ground Water Research and Territorial Allocation.** W. KOEHNE. *Deut. Wasserwirtschaft* **34**: 243 (Jun. '39). Among the first who considered the regular observation of ground water was Pettenkofer; after him, the Abbé Mendel

recorded two observations a week for 15 consecutive years. The supreme importance of ground water for agriculture and the closely interwoven relations existing between ground water and surface water are now recognized. Ground water conditions are now under observation at 20,000 stations in the Reich. Water has become a most precious asset, and control applied only to surface water is no longer thought adequate. Every drop of rain or flake of snow that falls will have to be husbanded. About 50 billions of cubic meters, that is 50 cubic kilometers, is quite a conservative estimate for the amount by which the store of ground water laid up in the Reich in winter is depleted every summer. The difference in quantity between the underground water reserve of the Reich when at its maximum and the same when at its minimum can not be less than somewhere between 150 and 200 cubic kilometers. Two series of maps are being brought out to exhibit ground water conditions throughout the Reich by means of contour lines, one set showing its depth below the surface, and the other, its elevation above sea level. It is suggested that localities, roughly speaking, fall into one or other of four classes, as regards their available supplies of ground water, namely, (1) none available, (2) enough only for agricultural requirements, (3) enough for factories, small-town supplies, or other such moderate demands, and (4) enough for a huge, full-size works of modern type. Doubt is expressed, however, as to whether any locality now-a-days could rank in class (3) or class (4) unless in addition to its ground water, it had also some surface supply available.—*Frank Hannan.*

**New Observations in Water Research.** MAX NUSS. Gas-u. Wasser. 82: 375 (May 20, '39). In the Rhine plain it was observed that the deeper the wells the less iron is in the water. A correlation was found between the ground water level and the rainfall in neighboring mountains. But the iron content of the upper layers seems to come from bogs in an old, filled-in river valley. An apparatus to take water samples from any desired depth in wells is described. It is used during the driving of test wells to find the most favorable depth for the screen. A method using measurement of electrical resistivity for the determination of the depth of water-bearing strata is also described.—*Max Suter.*

**Information on Ground-Water Temperature.** F. NÖTHLICH. Gesundheits-Ing. 61: 365 ('38). Ground water temperatures measured three times daily at the Teufelssee pump works, Berlin, from '25 to '37 are discussed. Av. depth of the 26 wells is 28.5 m. During the 13 yrs. 14,244 measurements made. Curves given of monthly averages, yearly avs. compared with the yearly avs. of the atmospheric temperature, and avs. for each month during whole period. Evident from the results that ground water temperatures are subject to variations both transitory and persisting over longer periods. Av. temp. of the ground water over whole period was 9.77°C. According to Koch, there is a rise of 10°C. for every 34 m. depth. Taking this into account the av. (8.96°C.) agrees closely with the av. atmospheric temperature of 8.88°C. for the same period.—*W. P. R.*

**The Use of Starch in Hydrological Investigations.** Department of Water Control (French). A. LE STRAT. Ann. Serv. tech. Hyg. (Paris) ('37). In

tracing the course of underground waters starch has the advantages of being colorless, non-toxic, and capable of identification in very small amounts by the purple color it gives with iodine. Fluorescein must be used in large quantities in order to facilitate detection; its presence in wells used for supply may cause complaints. In a preliminary experiment using rice starch, author found that the grains of starch in samples of water could be concentrated by using a gel precipitated from aluminum sulfate by adding ammonia, concentrations of  $10^{-8}$  of starch were detected by microscopic examination after concentration by alumina gel. A practical application of the method is described.—W. P. R.

### CORROSION OF PIPES

**Soil Corrosion of Cast-Iron Pipe.** L. T. BRYSON. '37-'38 Report, Board of Water Supply, City and County of Honolulu, T. H. The soft gray residue which is left as the corrosion product of graphitization of cast iron takes the place of the corroded iron. It is essentially a mixture of graphite, silica, and iron oxides. The hydrated iron oxide serves as a weak bonding material, holding the graphite and silica in place. This corrosion residue is quite permeable to water, and therefore not only fails to protect the underlying iron, but actually stimulates attack because it contains corrosive iron salts. Cracking of the pipe sometimes occurs, probably due to internal stresses set up by intercrystalline chemical changes. Microstructure of the metal undoubtedly has an important bearing on the susceptibility of c.i. to graphitization. Pit cast pipe shows a superiority in corrosion resistance over centrifugal pipe. Micrographic examinations of the metal show that the graphite is distributed in thin laminar form in the case of pit cast pipe; while there is a bunching or rolling of the graphite flakes into bundles in the case of centrifugal pipe, which is particularly noticeable in the unlined metal mold type. These varied forms of distribution of the graphite are attributed partly to the mechanical forces exerted by the centrifugal casting. As a result of the respective laminar or bunched distribution of the graphite upon chill and solidification of the metal, graphitization distributes itself more evenly over the surface of pit cast pipe; while deeper pitting and a softer and more porous residue is characteristic of the corrosion products found in centrifugal pipe. Analyses of the corrosion products showed that both oxidizing and reducing processes of chemical reaction took place concurrently. The predominating process however was reduction, indicated by the amounts of ferrous salts and by the evident decomposition of sulfate ions, with formation of sulfite and free sulfur. Soil colloids are important insofar as they control the deposition of a film of soil upon the pipe capable of excluding oxygen. The corrosion residues show distinctly the reducing effects of cathodic hydrogen, and that consequently air has had little or no access to the corroding areas. Semi-soluble sulfates, by hydrolysis, tend to maintain hydrogen ion concentration. The sum of water soluble sulfate and chloride ions, as calculated to the corresponding sodium salts, are considered a measure of electrolytic conductivity. A tentative interpretation of results found would indicate that clay soils must be practically free from soluble salts in order to be non-corrosive. *Laboratory Corrosion Tests:* Two series of laboratory tests were arranged, one using sulfate-free chlorides and the other using chloride-free sulfates as the electrolyte, to determine whether

one or the other is essential to the process of graphitization. Duration of the tests was 40 days, and all samples showed general surface graphitization, with scattered deeply attacked spots. Centrifugally cast specimens were used. All the materials used, including the clay and the solutions, were thoroughly sterilized. These tests showed that graphitization of c.i. can proceed without aid of bacteria, and that either sulfates or chlorides alone can serve as the electrolyte. Investigation of the electrolytic potential differences between iron and carbon was made, using a specially constructed laboratory test cell. The effects of current density, hydrogen over-potentials, and of polarizing films formed upon the electrode surfaces were investigated and observations reported. These experiments showed that in spite of partial cathodic polarization, the current density at the anode is sufficient to dissolve an appreciable amount of iron. The electrolytic potential differences in (approx.) absence of oxygen were determined by the difference between the cathodic over-voltage and the solution-pressure voltage of the iron. Gradual diminution of current, after the first three hours, was due to increasing concentration of dissolved iron. Bearing in mind the predominance of ferrous salts in the corrosion residues, the effect of aeration can be predicted. Access of a limited amount of oxygen will assist in depolarizing the cathode, further lowering its over-voltage and stimulating corrosion. This effect will be overshadowed if an ample amount of oxygen has access, because the main anode product, ferrous hydroxide, becomes converted into ferric hydroxide, a very bulky, insoluble precipitate with high adsorptive power. Polarization of the anode then takes place, the electrolyte is deprived of its ions by adsorption on the large surface of the colloidal ferric hydroxide, and corrosion ceases. The final result is a film of rust on the solid iron, instead of a pit hole filled with soft residue. Graphitized spots, in their characteristic reduced state, are anodic to the intact pipe surfaces, but after oxidation by exposure to air their polarity reverses. Stray current electrolysis will cause rapid penetrative pitting, leaving no corrosion residue. The high acid concentration built up in the pit probably dissolves the ferrous hydroxide as fast as it is formed, and consequently the liberated graphite and silica have no binding material to hold them in place. *Prevention:* (1) Perfect waterproof coatings will prevent corrosion. Difficulties lie in providing a coating having sufficient mechanical strength to resist abrasion by handling, backfilling, and by soil stresses. (2) Concrete is not impervious to water, but it answers the purpose by maintaining a high, well-buffered pH value on the pipe surface. It has the necessary mechanical strength, when properly applied. (3) Trench drainage and proper selection of backfill material that will permit free passage of air-saturated surface water will aid formation of an anodic film of ferric hydroxide on the exposed iron surfaces, which will inhibit or even stop the corrosion process. ★[Bryson's study of the processes of graphitization of c.i. actually support the electro-biochemical theory of von Wolzogen Kuhr. He shows that the bacterial actions of sulfate-sulfide reduction and cathodic depolarization are not necessarily essential to this distinctive type of iron corrosion process in the soil. Although not specifically mentioned, he shows indirectly by his analysis of "autoelectrolysis" how such bacterial actions can, and undoubtedly will, greatly accelerate the graphitization of c.i. He shows how any

source of supply of sulfate and chloride ions will stimulate the corrosion processes involved and greatly increase the corrosion rate. He shows the same to be true about any source of gradual and constant cathodic depolarization, such as that provided by living hydrogen-consuming bacterial organisms. Bryson evidently favors concrete coatings. He does not in this paper give his specifications for a sulfate-resisting cement concrete, nor does he discuss the processes of deterioration of ordinary cements by the chemical action of sulfate ions. He does not mention any of the after effects of aeration following sulfate-sulfide reduction by biochemical processes, resulting effects of which are to yield a dilute solution of sulfuric acid capable of attacking ordinary portland cement concrete. Presumably, however, he makes provision for retarding this phase of deterioration of the concrete used as a protective armor for c.i. wherever such is required.]★—Robert R. Ashline.

**Engineering Significance of National Bureau of Standards Soil-Corrosion Data.** KIRK H. LOGAN. J. Research Nat. Bur. Standards **22**: 109 (Jan. '39). Since beginning of investigation in '22 with 7000 specimens, program has grown by '38 to include burial of 33,000 samples of various pipe materials in 46 representative soils.  $\frac{3}{4}$  of specimens have been removed and examined. Found early that rate of penetration of pits in ferrous metals was not constant and that max. pit depth on large specimen was greater than on small specimen of same material. Tests on small pipe samples cannot, therefore, give definite information regarding larger pipe. Soil-corrosion tests should be expressed in terms of 3 factors: (1) inherent corrosiveness of soil, (2) effect of time on rate of penetration, (3) relation of area from which max. pit is measured to depth of that pit. Soil characteristics differ greatly, and corrosive soils are so widely distributed that no av. value for corrosiveness of soils, and no allowance or factor of safety based on av. soil conditions should be used in design of a specific pipe line. Necessary in each case to consider corrosiveness of particular soils to be encountered. Exptl. data and theory indicate pit-depth-area relation influenced by soil characteristics not yet detnd. Pitting data on hand suggests corrosion of wrought iron somewhat more uniform and that of pit-cast iron somewhat less uniform than that of steel. Two formulas have been set up to show relationship of max. pit depth to area from which it was selected: Scott's equation:  $P = bA^a$  in which  $P$  is max. pit depth associated with an area  $A$ , and  $a$  and  $b$  are constants which change for different conditions; and Ewing's:  $P = P_1 (C \log A + 1)$  in which  $P$  is same as above,  $C$  a constant varying with conditions; this later formula seeming to fit data better than Scott's. Relationship of  $P$  to  $A$  must be taken into account in correct interpretation of soil-corrosion data. Because rate of corrosion decreases with period of exposure, this relationship can be expressed well enough by a straight line equation which will approximate for most soils shape of curve beyond first few years of exposure. For purpose of paper Ewing's pit-depth-time equation has been combined with Scott's and Ewing's pit-depth-area equations giving respectively  $P = kT^a A^a$  and  $P = kT^n (C \log A + 1)$ , in which  $P$  is pit depth on area  $A$ , at any time,  $T$ , and  $C$ ,  $a$ ,  $k$  and  $n$  are constants derived from an analysis of pit depth data. General most applicable equation is:  $P_2 = P_1 (C \log A + 1) T^n$  in which  $P_2$  is av. of deepest pits on a number of

specimens having area  $A$  and  $T$  is age of specimens.  $P_1$  is pit depth on unit area exposed for unit time and  $C$  and  $n$  are constants which depend on soil conditions. Equations used to calculate av. depth of deepest pit on 1000 linear ft. of 8" pipe exposed for 30 yrs. in various soils—these results tabulated. Figures given also allow estimating length of time before puncture of pipe might be expected to occur. On the whole these figures show that more of the 47 soils tested are shown to be seriously corrosive than first thought. In general, correspondence of computed values sufficiently good to warrant conclusion, that if local conditions are properly taken into account, the results of soil-corrosion investigation properly adjusted for the effects of time and area constitute the best available basis for decisions as to proper wall thickness and the necessity for protection of a proposed pipe line. 10 comparisons of actual field data with computed figures given.—*Martin E. Flentje.*

**Correlation of an Electrolytic Corrosion Test with the Actual Corrosiveness of Soils.** IRVING A. DENISON AND ROBERT B. DARNIELLE. *J. Research Nat. Bur. Standards* **21**: 819 (Dec. '38). The corrosiveness of soils along a 128 mi. section of a system of 5 parallel 8" pipe lines was detn'd. by data from leaks and replacement of pipe and this information then compared with electrolytic test for measurement of corrosiveness of soils. Test consists in measurement of polarization voltage at various current densities of a specially designed corrosion cell with steel electrodes and electrolyte the soil under test. Corrosion shown to be closely associated with loss in wt. of anode of cell over 2 wks. period and also with long time U. S. field tests. Number of leaks per 10,000' of pipe during period of 33 yrs. and by percentage of line reconditioned used as measure of corrosion. Corrosiveness of soil decreases as drainage improves. Mean values of av. current density in potential range from 0 to 0.3 volt for 27 soil samples tabulated, varied from 0.6 to 15.8. In general, soil types showing highest values of current density are heavy in texture or poorly drained, or both. Tests then further compared with actual records from leaks and reconditioning, indicated that 19 of 27 soils (70%) showed at least fair correlation between electrolytic test and field experience. On 80% of actual length of pipe lines the electrolytic test would have correctly predicted relative corrosiveness of soil.—*Martin E. Flentje.*

**Determination of Soil Aggressiveness.** H. KLAS. *Vom Wasser* **13**: 120 ('38). Procedure should be: (1) determine the physical properties of the soils by mechanical analysis, (2) determine the chemical composition of the soils, in general only of the water soluble substances. The mechanical soil analysis must determine the structure of the soil, plasticity and the plastic index. Chemical analysis of water soluble portions from soil should include: pH, total acidity, conductivity, sulfide, combined nitrogens, and calcium carbonate. Brief methods given for procedure for various determinations.—*Willem Rudolfs.*

**Measurement of Corrosion.** ADOLF DEMSKI. *Gas-u. Wasser*. **82**: 341 (May 27, '39). A new method of measuring corrosion is described. It is based on the fact that at room temperature, the electric resistance of the

product of corrosion, is great compared to that of the metals. Details of a method are developed that allows the quantitative measurement of the progress of corrosion by determining the increase in the resistance occurring during corrosion. The limits of sensitiveness and the pitfalls of this method are discussed and data from actual tests are given.—*Max Suter.*

**Micro-biological Experiments in Anaerobic Corrosion.** H. J. BUNKER. J. Soc. chem. Ind. (Br.) **58**: 93 ('39). Discusses, with reference to the literature, the role played by sulfate-reducing organisms of the type *Vibrio desulphuricans* in the corrosion of metals under anaerobic conditions. Examination of material from corroded pipes and reproduction of this type of corrosion in the laboratory are described. The suggested mechanism of corrosion postulates the production of metallic sulfide near the pipe. A table showing the sulfide content of materials taken from a 24" steel main (contents of pits, particles of the bitumen and hessian wrappings, clay, soil, etc.) shows that up to 0.66% of sulfide was present. Max. amount came from a pipe wrapped in hessian. Such wrapping appears to be undesirable since it forms a source of food for bacteria able to decompose cellulose; the products provide nutrient material for sulfate-reducing bacteria in the immediate vicinity of the pipe. Treating the wrapping with bitumen does not offer any additional protection. In comparing the activity of sulfate-reducing bacteria, the rates of reduction of sulfate in liquid cultures containing lactate were observed. Bacteria isolated from corroded pipes were examined by this method. In some cases the organisms from the surface of the pipe were more active than those from the surrounding soil. In the laboratory, corrosion similar to that found in practice was caused by sealing a strip of iron in a glass vessel containing a medium of sulfate and lactose inoculated with a pure culture of *Vibrio desulphuricans*; incubation at 30°C. for several weeks resulted in graphitic corrosion of the specimen. Quantitative experiments showed that, under these conditions, a disc of mild steel 25 mm. diam. and 6 mm. thick, lost 0.3774 grams weight in 156 days; a control test without bacteria resulted in loss of only 0.0194 grams from similar specimen; max. penetration in the first disc was 0.23 mm. Apparatus illustrated. Loss of weight of the control was probably due to ammonium chloride in the medium. In an experiment with a medium which contained no ferrous ammonium sulfate the loss of weight by the specimen in 22 weeks was only 5 mg. Further experiments in progress.—*W. P. R.*

**A Biochemical Investigation of the Tuberculation of Water Pipes.** S. C. PILLAI. Proc. Natl. Inst. Sci. India **4**: 295 ('38). The attack of dissolved gases and other corrosive elements takes place where points of bare Fe are exposed. The present practice of dipping and coating the pipes with protective material is defective. The desired surface finish is not achieved, so that the necessity of devising improved methods of impregnating the pipes with protective material is stressed. The tubercles collected from the particular system are more or less similar in their chem. compn.  $\text{Fe}^{+++}$  is the chief constituent, while Al,  $\text{SiO}_2$ ,  $\text{Fe}^{++}$ , Ca, Mg, Mn and P are present in much smaller proportions; org. C and N are also present in small quantities. The Fe necessary for the formation of the tubercle comes mostly from the pipe itself, while

the water passing through the pipe system supplies the other constituents. It is more than likely that a large proportion of the Si, P, Mn and C is derived from the pipe, for c.i. contains these in appreciable quantities. There is a slight but progressive increase in the Fe content of the water as it passes through the pipe. The tubercles examd. contain none of the better known Fe bacteria, but they are invariably assocd. with a lower form of fluorescent bacillus of the *Pseudomonas* type which is derived from the water. The bacterium has been completely characterized and found to be a new species, *Pseudomonas ferrugineum*, which lives in the rust. The function of this organism in the formation of tubercles is that it gives to the tubercle the characteristic nodular appearance, the necessary strength and compactness to withstand the current of water thus enabling it to exist indefinitely in the pipe; it also facilitates oxidation of  $\text{Fe}^{++}$  to the  $\text{Fe}^{+++}$ . Tuberculation is always preceded by corrosion of the Fe. The dissolved O,  $\text{CO}_2$  and other corrosive elements, such as Cl, attack the Fe of the pipe. A practical method for preventing formation of tubercles takes advantage of the specificity of the bacterium which is found in assocn. with the tubercle and which, to a large extent, is responsible for the development of the tubercular structure out of the corrosion products. The organism flourishes best at pH 6.8-7 and it fails to grow when the pH of the medium is about 9.  $\text{Ca}(\text{OH})_2$  is recommended for maintaining the desired pH.—C. A.

**Explanation of the Langelier Method for Determining pH Saturation and the Saturation Index.** CHARLES P. HOOVER. Ann. Rept. Ohio Conf. Water Purif. 18: 19 ('38). Also published in J.A.W.W.A. 30: 1802 (Nov. '38). Discussion. R. D. SCOTT. p. 22. Finely ground marble is preferable to c. p. calcium carbonate for "marble test." If latter is employed, it should be tested for free alkali content. Fineness is important. Bottle containing water-calcium carbonate mixture should be completely filled and pH determined on decanted supernatant: filtration alters the pH. Ten min. shaking with calcium carbonate is considered ample. Silica content of water affects pH value calculated by Langelier formula: about 0.05 should be added for each 17 p.p.m. Calculated pH may be 0.05 too high on zeolite-softened water or natural waters containing high concentrations of alkali bicarbonates. Temp. is important in both methods.—R. E. Thompson.

**The Continuous Stability Indicator.** LINN H. ENSLOW. W. W. and Sew. 86: 107 (Mar. '39). Described a device for continuously providing the marble test and Langelier corrosivity index by flow of water through glass tubing filled with chalk and sand. Samples of water from the influent and effluent are tested for alkalinity and pH. Final pH indicates desired pH in plant effluent. Device is adjusted to give about 2 hours contact with chalk. Ibid. 86: 283 (Jul. '39). Comments by various authors on use and interpretation of results from continuous stability indicator suggested by Enslow. Results found to be not strictly comparable to the Langelier index. Test is found to be of practical value to plant operators and details for its use are suggested.—H. E. Hudson, Jr.

**Problems of Corrosion and Materials in Hot Water and Heating Plants.** L. W. HAASE. *Gesundheits-Ing. (Ger.)* **62**: 86 ('39). Discusses advances made in methods of preventing corrosion in hot-water and heating plants since '35 when he gave an account of the position. Great developments in use of copper have taken place. In plants using hot water it is no longer essential to use copper; it may still be the simplest and often the most economical material, but users should familiarize themselves with new materials and methods and test them in practice. Methods of prevention of corrosion include methods of construction and chemical and physical treatment of the water. Advantages and disadvantages of low-pressure and high-pressure hot-water plants are discussed. In plants with pre-heaters, heating coils, or counter-current apparatus of copper, the danger lies not, as is often assumed, in the contact of different metals but in the composition of the cold water. Most waters will dissolve copper to some extent and very small amounts of copper may suffice to attack iron by the formation of local elements. To avoid corrosion in hot-plants with copper parts, water should be used which when cold has no aggressive action on iron or galvanized iron and which can quickly form an effective protective layer. All other waters require treatment. In hot water the lime-carbon dioxide equilibrium is not of the same importance as in cold water but the amount and state of solution of oxygen in the water determines the extent of attack. Removal of oxygen by the Groeck process has proved an effective method of preventing corrosion; the process has been improved by the use of catalysts to accelerate the action. Of methods of treatment with chemicals, treatment with phosphate is the only method now employed. Object of this treatment is to deposit a protective layer between the metal surface and any harmful constituents of the water. Phosphate treatment in hot-water plants differs from that in boiler operation as there is in the former no intention of softening the water. It was at first thought that by a sufficiently large addition of phosphate, a coating of alkaline earth phosphates would be deposited in the pipes; it was found however that protection was more certainly achieved by adding small amounts (5-7 mg.  $P_2O_5$  per liter) of phosphate; the double salts which are then formed are almost completely insoluble in water with a pH range of 5-9 and are formed even in very soft waters. When a protective layer has been formed on iron by this method, untreated aggressive water can be passed over it for long periods without attacking the iron. Treatment of the water by colloids was found to be successful only under exceptionally favorable conditions; this method had serious disadvantages and has generally been abandoned. Electrolytic methods have not been tried for periods long enough to judge their efficiency; the effect in these methods is not electrical but is due to an increase of alkalinity by the solution of metal and to reduction of the content of aggressive oxygen in the water; the dissolved metal may also be of assistance in forming a protective layer. The extent to which protection would be afforded to the more distant parts of the system by this process is not known. The "Magno" process is being increasingly adopted in treating hot water. The Magno material removes the carbon dioxide freed by the rise in temperature of the water; with reduction in temperature through the system the lime-carbon dioxide equilibrium alters and a protective layer is deposited. It has been found however that treatment of the cold water in a

Magno filter suffices in some cases to prevent corrosion when the water is heated. The main action is therefore mechanical filtration which, at a suitable rate of filtration, will remove solid and colloidal substances which would lead to the formation of local elements. It appears thus that waters which are not aggressive but contain more than a certain quantity of solid or colloidal matter can be made suitable for use in hot-water plants by filtration through sand; if, however, the water is aggressive, filtration and removal of aggressive properties can be combined by using a Magno filter. In new standards for hot-water plants being prepared by the German Standards Committee the process of oxygen removal, phosphate treatment and filtration are to be included. Author then deals with protection by choice or treatment of materials, discussing the efficiency of porcelain, artificial resin, "Kuprema" and "mipolam" pipes, and coatings of artificial resin, (See abstracts J.A.W.W.A. 29: 2054 (Dec. '37); 30: 1252 (Jul. '38)), copper plate, and rustless steel plate. Apart from the use of the new rustless steel "Remanit" (See abstract J.A.W.W.A. 30: 1429 (Aug. '38)), no generally satisfactory solution to the problem of corrosion-resistant material has been found. In dealing with heating plants, author discusses changes in construction and methods of operation of water and steam heating plants which have made failures due to corrosion increasingly common in recent years; these include the use of thinner material in construction and the use of superheated water under pressure. There is always some leakage of oxygen into the circulating water and the constant heating removes practically all hardness so that methods for protection of the metal are necessary. Methods used for the treatment of boiler feed water (treatment by soda or phosphate or removal of oxygen by "Rostex" filters) are partially but not completely effective. The plant must either be completely enclosed to prevent leakage of oxygen or reagents which have a direct protective action on the metal, regardless of temperature or the composition of the water, must be used. Such a reagent is chromic acid. Diagram is given of apparatus by which chromic acid in 20-50% solution can be mixed with feed water. Though hot water and low pressure steam heating plants can be protected by chromic acid, protection of the condenser pipes in steam plants is not generally achieved although in some cases the condensate water has been found to contain chromic acid so that the metal is protected. In the preparation of protective materials for condenser tubes, iron pipes coated with artificial resin (Heika pipes) have been successfully employed experimentally for steam at temperatures up to 145°C.—*W. P. R.* (See also abstract J.A.W.W.A. 30: 1748 (Oct. '38)).

**Chemical Processes in Deacidification of Water by Magno.** W. PLUCKER. *Zeitschrift f. Untersuchung Lebensmittel* 77: 50 (Jan. '39). Magno is made from dolomite by heating it to 500°C. Most of the magnesium carbonate is changed to magnesium oxide, but the calcium remains as a carbonate. Results of laboratory experiments on the rate of CO<sub>2</sub> removal and the increase in hardness at different depths of filter material, and different rates of filtration, are given. Carbonate hardness mainly due to magnesium increases with the time of contact. Complete iron removal requires over 28" depth of Magno-mass. Iron is removed by the high pH of 10 formed on the surface of the

Magno grains. Results are also given from several plants using Magno filters, whereby it was found that the increase in hardness due to calcium is greater than that due to Mg, which disagrees with the laboratory test. The reason for this difference is not yet found. Protective coatings are formed in distribution systems following Magno treatment. The taste of the water does not seem to be changed by the increase in magnesium salts.—*Max Suter.*

**Deacidification at the Water Works of Frankfurt a/M.** REINHOLD BRÜCHE. *Gas-u. Wasser.* **82**: 264 (Apr. 22, '39). Frankfurt a/M is supplied by water rich in free CO<sub>2</sub>. The methods of treatment vary depending on the source and plant. In one plant a marble filter with a grain size of from 3 to 6 mm. completely removes 7 to 8 p.p.m. CO<sub>2</sub>. In another plant a Magno filter (burned dolomite) reduces 30 p.p.m. CO<sub>2</sub> to below 5 p.p.m. One mg. CO<sub>2</sub> causes a loss of 1.3 mg. Magno-mass, increasing the hardness from 18 to 72 p.p.m. To treat 1 m.g.d. of water of such composition requires a filter containing 62 short tons of Magno-mass. Refilling with Magno-mass has then to be done about every two months. Lime is used in two plants. This requires constant control, due to variations in pumpage. Aeration removes 30 p.p.m. CO<sub>2</sub> on one supply which contains 55 p.p.m. CO<sub>2</sub>. As this water contains 215 p.p.m. of hardness it is not advisable to use chemical treatment. First cost and operating costs are the highest with the Magno filter and the least with lime treatment, even if compared to a uniform CO<sub>2</sub> content. The reduction to a CO<sub>2</sub> content of less than 5 p.p.m. seems to be enough to allow formation of an iron-lime coating for pipe protection.—*Max Suter.* (See also abstract J.A.W.W.A. **30**: 2106 (Dec. '38)).

**Summary of Experimental Work at Lawrence Experiment Station for the Year 1938.** ANON. Massachusetts Dept. Pub. Hlth., Div. San. Eng., News Letter (Mar. '39). "Hydrochloric acid to which an inhibitor has been added is sometimes used for removal of rust and scale from iron pipe. Based on the amount of iron dissolved from rustfree metal by uninhibited acid, the inhibited acid removed, under the same conditions, 53% as much from steel, 63% as much from wrought iron and 77% as much from cast iron. With lead no difference was noted."

**Cathodic Protection of Pipe Lines.** ANON. J. Franklin Inst. **228**: 93 (Jul. '39). *Notes from the Nat. Bur. Stds.* Use of cathodic protection has rapidly increased since the first installation on gas transmission lines in '30, until now practically all large companies are using this method to prevent corrosion of parts of their pipe line systems. Cathodic protection usually installed on lines that were properly coated before they were laid. By applying current cathodically the potential of the entire pipe line is lowered with respect to a reference half-cell in the surrounding soil so that current flows toward the pipe through any imperfections in the coating. However, by careful design and proper location of anodes from which the drainage current is discharged into the soil, it has been found to be economical in certain cases to apply cathodic protection to bare lines such as gas pipe lines of large diameter. On bare oil lines, which are of smaller diameter, reconditioning operations still seem to be preferable to cathodic protection if the occurrence of leaks should become

excessive. Reference is made in detail to a paper *Application of Electrochemistry to Cathodic Protection*, by SCOTT EWING, before the Petr. Ind. Elect. Assoc., May 4, '39. In this paper electrochemical principles were discussed which will be useful by helping make more accurate voltage measurements and by giving better understanding of the corrosion process. Design of copper-copper sulfate and cadmium-cadmium sulfate half-cells has been improved to correct some of the faults of the half-cells previously used. Brief description of these improved half-cells was given, together with temperature correction curves. With these half-cells the potential difference between two points in the soil can be measured with an error not greater than 2 mv. Consideration of potentials and currents in a corroding pipe line indicates that in some cases it should be possible to apply cathodic protection to bare pipe lines of small diameter. The protective current should be drained from the pipe in places where the reference half-cell has a large positive potential with respect to the pipe. In such places the pipe is usually found to be discharging current into the soil. By locating anodes so as to take advantage of the galvanic currents in the pipe, a much greater length of line can be protected with one anode. The potential of the pipe with respect to a reference half-cell is not appreciably changed except in the originally anodic area near the anode. This is probably the only part of the line that really needs protection. The feasibility of the method depends upon the distribution and stability of the originally anodic areas along the pipe line. Whether it is economical or not will depend upon the relative costs of cathodic protection and probable future reconditioning.

#### CHEMICAL ANALYSIS OF WATER

**Determination of Small Quantities of Heavy Metals in Waters.** E. ABRAHAMCIK. *Mikrochemie* (Vienna) **25**: 228 ('38). For detg. small quantities of heavy metals in samples of water, the tedious evapn. can often be avoided by filtering the water, at the place where the sample is taken, through a permutite tube. A simple arrangement is shown in which the permutite tube is furnished with a separable funnel top and stopcock bottom. The top and bottom parts can thus be shifted from one tube to another. Then, in the lab. the adsorbed heavy metals can be washed out with NaCl soln. Instead of permutite, a resinous filtering material is obtainable on the market. Ions of Cu, Ag, Au, Zn and Pb can be removed from very dil. solns. by treatment with dithizone in  $\text{CHCl}_3$ . The dithizone complexes can be dissolved in the  $\text{CHCl}_3$  by shaking. In many cases excellent recovery of added ion was accomplished.—C. A.

**Analytical Separation of Cations by Extraction.** S. STENE. *Tidsskr. Kemi Bergv.* **19**: 6 ('39); *Chem. Zbl.* **1**: 3433 ('39). In the analysis of potable water, calcium, magnesium, and zinc can be separated from other cations in the water by converting them to insoluble compounds with acetyl acetone and extracting these with a suitable solvent such as carbon tetrachloride or chloroform. At pH values 4.5-7.5, iron, copper, aluminum, beryllium, and cerium can be separated in this way from calcium, barium, magnesium, zinc, and other metals. Ferrous and cuprous ions cannot be extracted by this method.—W. P. R.

**Colorimetric Determination of Zinc in Water With Dithizone.** GAD, GEORG AND KÄTE NAUMANN. *Gas-u. Wasser.* **82**: 168 (Mar. 11, '39). Diphenylthiocarbazone, shortly called dithizone, forms with metals strongly colored compounds which are soluble in organic solvents. However, the compound formed with zinc is also soluble in slightly alkaline water. This property is used in the following method for determination of zinc. To 50 ml. of the water to be tested add 0.5 ml. of 10% Rochelle salt solution; after shaking add 1.0 ml. of 20% potassium bicarbonate solution and after mixing, 0.5 ml. of 0.05% alcoholic solution of dithizone. Similarly treat solutions with a known content of zinc ranging from 0.02 to 1.00 p.p.m. The colors formed go with increasing zinc content from yellow through orange to bluish red. The high sensitivity of the test often allows the use of only 5 ml. of the water diluted to 50 ml. In this case the test is not influenced by up to 10 p.p.m. Fe, up to 10 p.p.m. Pb and up to 3 p.p.m. Mn. Copper has an influence on the test if present over 1 p.p.m. Methods are given to avoid this influence.—*Max Suter.*

**A New Color Reagent for Lead and Its Use as an Indicator in the Titration of Various Cations and Anions.** B. S. EVANS. *Analyst* **64**: 2 ('39). Gives directions for preparation and use of a new reagent for volumetric determination of small amounts of lead, zinc, vanadates, molybdates, phosphates, arsenates and tungstates. The reagent is prepared by adding exactly 2.0 ml. of nitric acid (sp. gr. 1.2) to a mixture of 10 ml. of 1.5% solution of diphenylcarbazine in alcohol and 30 ml. of pure pyridine, diluted with 120 ml. of cold water. The liquid is allowed to stand overnight. It is deep orange-brown in color; with lead and zinc it gives a red color; the colors produced with other ions are summarized in a table. In the determination of lead, a solution of the lead as nitrate is made neutral to litmus with ammonia and diluted to 100 ml. with distilled water; 10 ml. of the reagent and 30 ml. of acetone are added and the solution is titrated with a dilute aqueous solution of phosphoric acid until the color of the solution matches that of 10 ml. of reagent and 30 ml. of acetone in 100 ml. of distilled water. Acetone eliminates the red tones in the reagent without affecting the color it gives with lead, thus making the end-point easier to recognize. Amounts of lead nitrate varying from 0.01 to 0.001 gram were determined satisfactorily both by the above method and by titration with ammonium vanadate; 1 ml. of a solution containing 0.00008 gram vanadium per ml. corresponded to 0.000723 gram of lead nitrate. In determination of phosphates, 100 ml. of the neutralized solution mixed with 10 ml. of the reagent and 30 ml. of acetone is titrated with lead nitrate solution (1 gram per liter); the end point is a faint red color. Methods for the determination of molybdates, vanadates, arsenates and tungstates are also described. In the determination of zinc, 1 ml. of a 20% solution of pyridine is added to the faintly acid solution of zinc, which is diluted with distilled water to 100 ml.; 30 ml. of acetone, 5 ml. of reagent and 5 ml. of chloroform are added and the liquid is titrated with M/1000 potassium ferrocyanide with vigorous shaking until the chloroform shows only the color of the reagent. Traces of nickel produce a permanent pink color which may be counteracted by adding 2 drops of a 10% solution of potassium cyanide to both sample and blank before adding pyridine. Copper, citrates and tartrates interfere with the color produced by lead and the reagent.—*W. P. R.*

**Determination of Lead in Drinking Water.** H. INGLESON. *Analyst*. **63**: 546 (Aug. '38). Average lead content of water under normal domestic use is obtained by absorbing lead in calcium carbonate—magnesium oxide filter fitted to tap. After known volume of water (sometimes as much as 245 gals.) passed through filter (10% MgO, 90% CaCO<sub>3</sub>) absorbent material is dissolved in minimum amount of nitric acid, solution is made alkaline with ammonia and lead and copper precipitated with potassium chromate and filtered. Precipitate is dissolved in dilute hydrochloric acid, taken to dryness repeatedly with concentrated hydrochloric acid and extracted with hot water. Residue added to material insoluble in nitric is treated with hydrofluoric acid to remove silica and added to lead solution. Lead and copper are precipitated with hydrogen sulfide, precipitates dissolved in nitric acid and lead determined by anodic deposition.—*W. G. Carey.*

**Determination of Iodine.** A. C. BRATTON AND J. F. MCCLENDON. *Ind. Eng. Chem.-Anal. Ed.* **10**: 600 (Oct. '38). Method reported is characterized by reduction of quantities of reagents to a min. and is adapted to detn. of I in drinking water. Natural water samples are made alkaline with NaOH, 10–20 mg. of sodium azide added per liter, and then evaporated to dryness and fused with a trace of rare earth oxides (as oxidation catalyst) before transferring to the still. Chlorinated waters are evaporated and fused before adding the sodium azide (to destroy nitrite and iodate). To the still contg. the sample in about 3 cc. volume are added 3 cc. of 21 N H<sub>2</sub>SO<sub>4</sub> and 0.2 cc. of 4% ferric sulfate and the liberated I distilled with an air stream into 2 cc. of Br water (where it is oxidized to iodate) for 2 min. after SO<sub>3</sub> fumes appear. A trap must be used to prevent carrying iron into the distillate. The test tube contg. the distillate is heated in boiling water, the excess Br blown out with an air current, the tube cooled in melting ice, 10 mg. of KI added and the iodate titrated with 0.001 N Na<sub>2</sub>S<sub>2</sub>O<sub>3</sub> soln. from a 1 cc. microburet graduated to 0.001 cc. Endpoint detd. electrometrically with a conen. cell. 0.04% of iodate I may be titrated, with proper correction for reagent blanks. Using 200 cc. samples made alk. with 0.1 gram of NaOH, and following above method, 2.15 parts per billion of I were found in Minneapolis tap water, and 2.95 p.p.b. in LaGrange deep well water. Method is shorter and less expensive than combustion methods, and the electrometric titration is adapted to smaller quantities of I than heretofore. Accuracy for water samples is estimated at 2%, based on recovery of added I.—*Selma Gottlieb.*

**The Direct Determination of Ammonia in Water.** J. C. HARRAL. *Analyst* **63**: 597 (Aug. '38). In Kitto's method for above (see abstract—"A Contribution to Water Analysis", *J.A.W.W.A.* **30**: 1737 (Oct. '37)) chloride over 50 p.p.m. interferes. Chloride with persulfate and acid yields free chlorine which reacts with ammonia present to give nitrogen chloride. Water with 180 p.p.m. chloride caused complete loss of ammonia but when diluted 1:10 with ammonia free water gave alb. ammonia figure identical with distillation. Manganese also interferes because it oxidizes tartrate to aldehyde which gives color with Nessler. To such water crystal of oxalic acid is added to warm solution after persulfate, and some Rochelle salt just before Nesslerising.—*W. G. Carey.*

**Determination of Traces of Manganese in Natural Waters.** V. T. CHUÍKO. Zh. prikl. Khim. (Leningr.) **11**: 530 ('38). In determination of manganese in natural waters, alkali, to give a pH value of 9, and magnesium sulfate are added. When the magnesium hydroxide has been completely precipitated, the clear liquid is decanted and the precipitate is dissolved in sulfuric acid. Manganese is then determined colorimetrically. Manganese suspended in the water can be separated from dissolved manganese by coagulation with potash alum at a pH value of 7.6-8.4.—W. P. R.

**Detection of Arsenic in Potable Waters.** JÁNOS CSABAY AND ISTVÁN TANAY. Ber. ungar. pharm. Ges. **15**: 91 ('39). To 10 cc. of sample in a 100-cc. Kjeldahl flask add 5 cc. concd., As-free  $H_2O_2$  and 5 cc. concd. sulfuric acid (d. 1.8). Heat the flask carefully until the vol. of the contents diminishes to 8-10 cc. Cool, add 2 cc. fuming  $HNO_3$ , heat carefully again until nitrous gases cease to appear, cool, add 2-5 cc. portions of concd.  $H_2O_2$ . Heat again until soln. is evapd. to 5 cc. This procedure requires about 20 min. Now dil. the soln. with 30 cc. distd. water, add some particles of As-free Zn and one drop of a 5% soln. of  $CuSO_4$ . Put a loose cotton stopper in the neck of the flask to retain liquid drops taken by the gas current, bind a strip of filter paper through the opening of the flask and drop satd. aq.  $AgNO_3$  soln. on the filter paper. In the presence of As, a lemon-yellow spot appears, owing to formation of  $AsAg_3 \cdot AgNO_3$  (test of Gutzeit, Pharm. Ztg. **24** (1879)). The limit of sensitivity is 1  $\gamma$  As in 10 cc. water, i. e., 0.1 mg. in 1000 cc. Since lower As contents are not toxic the reaction is precise enough for practical purposes. Sol. compds. of Sb and  $H_3PO_3$  and  $H_2SO_4$  give similar reactions. Sb, however, cannot be found in natural waters, the acids are formed to phosphoric and sulfuric acids during the procedure and do not disturb the test.—C. A.

**Silica in Water.** M. C. SCHWARTZ. La. State Univ. Bull. **30**: 14: 1 ('38). The Schwartz technique is described and elaborated as follows: To a 50-ml. sample of clear water free from Fe and phosphate, add 2 ml. of  $(NH_4)_2MoO_4$  soln. and 1 ml. of 1:1 HCl. Mix and compare after 10 min. with a glass standard in a 100-mm. colorimeter. Check the  $SiO_2$  content of the reagents by repeating the same on 50 ml. of  $SiO_2$ -free distd.  $H_2O$ . Calibration methods of colorimeters are described.—C. A.

**A Syringe Pipet Method for the Determination of Oxygen in the Field.** R. J. WHITNEY. J. Exptl. Biol. (Br.) **15**: 564 ('38). A new method for the detn. of the  $O_2$  dissolved in natural waters. The method is quick, the whole analysis taking 10 min. It is accurate since contamination of the sample with atm. air is avoided at all stages. Only 10 cc. of  $H_2O$  is required for the analysis. The method consists of taking the sample, reagents are drawn into the syringe, and an amt. of  $I_2$  is liberated in the syringe which is chemically equiv. to the  $O_2$  that was originally present in the  $H_2O$  sample. The  $I_2$  soln. is titrated while still in the syringe by drawing in the  $Na_2S_2O_3$  soln. until the blue starch color is discharged. If the barrel vol. and the other consts. for the syringe are known, the  $O_2$  concn. of the sample is calcd. from the titer. The method is quicker to operate and more accurate than the ordinary methods of analysis.—C. A.

**Colorimetric Method for Estimation of Dissolved Oxygen in the Field.** M. L. JOHNSON AND R. J. WHITNEY. *J. Exptl. Biol.* **16**: 56 ('39). A sample of  $H_2O$  is transferred to a 100-cc. bottle, the usual precautions to avoid contact with air being taken. Two cc. of soln. contg. 10 grams KI and 34 grams NaOH per 100 cc. are run into the bottom of the bottle. 0.4 cc. 40%  $MnCl_2$  soln. is added and the mixt. shaken. The ppt. settles and 0.4 cc. concd.  $H_2SO_4$  is added. If the  $H_2O$  contains less than 4 cc.  $O_2$  per liter, 10 cc. of the  $I_2$  soln. is mixed with an equal vol. of  $CHCl_3$ , shaken for 30 sec. and transferred to a comparator tube. When the  $CHCl_3$  has sepd. out from the  $H_2O$ , its color is compared with disk A (0-2.0 cc./liter) or B (1.8-4.4 cc./liter) in the comparator. A modification of the Winkler method is described with which the  $O_2$  content of  $H_2O$  can be detd. quickly and without standard solns. with results satisfactory for ordinary purposes.—C. A.

**Effect of Ions on Mohr Method for Chloride Determination.** R. T. SHEEN AND H. L. KAHLER. *Ind. Eng. Chem.-Anal. Ed.* **10**: 628 (Nov. '38). Study was made of effect of ions commonly found in boiler waters on detn. of Cl with silver nitrate using potassium chromate as indicator. In range of 0 to 10 p.p.m. of Cl, representing condensates and feed waters, no interference was caused by 328 p.p.m. of  $SO_4$ , 400 p.p.m. of total alkalinity as  $CaCO_3$ , 600 p.p.m. of total hardness, 20 p.p.m. of  $PO_4$ , 400 p.p.m. of silica and 250 platinum units of color. In range of 10 to 1000 p.p.m., representing boiler salines, no interference was caused by 3000 p.p.m. of  $SO_4$ , 100 p.p.m. of total hardness, 400 p.p.m. of silica, 160 p.p.m. of  $PO_4$ , 40 p.p.m. of ferric Fe, 2000 p.p.m. of total alkalinity, and approx. 10,000 platinum units of color obtd. from tannin. However in both Cl ranges, sulfite interfered with Cl detn., causing high results. In presence of 200 p.p.m. of sulfite, detn. of 400 p.p.m. of Cl was 10% high. To eliminate effects of sulfite, neutralize sample to pH 4.3 (methyl orange endpoint), add 2 c.c. of hydrogen peroxide (3% by volume), stir, and reneutralize to alkaline side of methyl orange, and titrate as usual with  $AgNO_3$  and  $K_2CrO_4$ . Optimum range for titration is pH 7.4 to 10.8. Titration with  $AgNO_3$  of samples previously titrated with acid to methyl orange end-point is satisfactory because  $K_2CrO_4$  raises pH from 4.3 to 7.7.—Selma Gottlieb.

**Direct Titration of Sulfate.** WALTER V. BURG. *Ind. Eng. Chem.—Anal. Ed.* **11**: 28 (Jan. '39). Using 50 ml. sample, contg. 0.05 to 0.19 gram of  $SO_4$ , in 250 ml. Erlenmeyer flask, make just acid to phenolphthalein with approx. 0.02 *N*  $HNO_3$ . Add 16 ml. of absolute ethyl alcohol (95% alcohol may be used) and 14 drops of a 1% aqueous soln. of erythrosin B, and mix well. Temp. should not be over 30°C. Titrate with 0.1 *N* lead nitrate soln., running in at steady dropping rate with constant swirling until increasing persistence of violet color after each drop indicates approach of endpoint. Continue very slowly and with vigorous shaking to distinct violet in whole mixture. At endpoint drop of lead nitrate no longer causes dark spot on surface of mixture. Average relative error is less than 0.3% using samples prepd. from potassium and sodium sulfates. Chloride interferes but can be removed from soln. by detn. with  $AgNO_3$  using 0.1% aq. soln. of Na fluoresceinate or Na dichlorofluoresceinate as indicator. 50 ml. sample, contg. 0.04 to 0.31 gram of Cl and 0.10 to 0.38 gram of  $SO_4$  is adjusted with  $HNO_3$  as above, 4 to 7 drops of Cl indicator

added, and titrated with  $\text{AgNO}_3$  in diffuse light until ppt. turns reddish. Make vol. to 100 ml. with distd. water, filter, discarding first 5 or 10 ml. of filtrate, and detn.  $\text{SO}_4$  on 50 ml. of filtrate as directed above.—*Selma Gottlieb.*

**Rapid Field Methods for Determination of Hardness and Sulfate Content of Soil Waters and Potable Waters.** SZILÁRD PAPP. Magyar Chem. Folyoirat **44**: 162 ('38). Fill with the sample to the 100-cc. mark a 200–250-cc. flask graduated at 100 cc. Add from a 25-cc. cylinder in small portions a specially made K palmitate soln. until a red color is produced. The no. of cc. of palmitate soln. used must be doubled to obtain the hardness of the water in German degrees. The palmitate soln. is made as follows: Weigh about 20 grams K palmitate with 1 gram phenolphthalein and dissolve in 800 cc. pure MeOH with slight heating. Add 4% MeOH-KOH until a red color appears and then make the soln. up to 1 liter. (The soln. obtained is about 0.078 N). Sulfate can be detd. in the same sample. To the soln. after hardness detn. add 20 cc.  $\text{BaCl}_2$  soln. (contg. 9.53 grams cryst.  $\text{BaCl}_2$  in 1000 cc. water). Shake, let stand for 5 min. and add the special palmitate soln. until the red color once more appears. The no. of cc. of palmitate soln. used must be subtracted from 20 and the result multiplied by 30 to obtain the  $\text{SO}_4$  content of the water in mg./liter.—*C. A.*

**The Alizarin Test for Aluminum in Microchemistry.** OSCAR R. CHIODI. Rev. facultad cienc. quím. (Univ. nacl. La Plata) **12**: 117 ('37). Feigl's technique for detecting Al, modified by addn. of ether, permits detection of 1  $\gamma$  of Al. To soln. in test tube, add 2 drops alc. soln. of 0.1% alizarin and few drops of concd.  $\text{NH}_4\text{OH}$ . Shake and let stand for 15 min. Add 50% AcOH up to pH 5.2; add equal vol. of ether; shake and let stand. Violet color is found at boundary between aq. and ether phases.—*C. A.*

**Colorimetric Titration of Small Amounts of Fluoride with Thorium Nitrate.** D. DAHLE, R. U. BONNAR AND H. J. WICHMANN. J. Ass. off. agric. Chem., Wash. **21**: 459 ('38); Analyst **63**: 851 ('38). Describes a method for estimating up to 50  $\gamma$  of fluorine, applicable to the distillate obtained by distilling minerals containing fluorine with perchloric acid. Errors in titration which are caused if considerable amounts of perchloric acid are present in the distillate should be avoided by distilling below  $140^\circ\text{C}$ . 40 ml. of the sample solution are placed in a 50 ml. Nessler tube and 40 ml. of distilled water are placed in another similar tube. 1 ml. of a 0.01% aqueous solution of sodium alizarine sulfonate is added to each, and 0.2% sodium hydroxide solution is run in to the sample until a faint pink color matching that in the other tube is obtained. Exactly 2 ml. of 0.05N hydrochloric acid are added to each tube and thorium nitrate solution, containing about 0.25 gram per liter, is added to the sample from a burette until a faint pink color appears on mixing. The same volume of thorium nitrate solution is added to the other tube, the contents of which are then titrated with standard sodium fluoride solution (1 ml. = 0.01 mg. of fluorine) until the color matches that of the sample. The two solutions must be diluted to the same volume before final comparison is made. The fluorine content of the sample then equals the amount added to the titration tube.—*W. P. R.*

**A New Method for the Colorimetric Determination of Fluorides.** V. P. SHVEDOV. *Lab. Prakt.* (U. S. S. R.) '39, No. 2-3: 22. Following indicators were used: flavopurpurin, anthragallol, alizarinecyanine, cerulein. The 1, 2, 4, 5, 8-alizarinecyanine was found most sensitive, giving max. sensitivity of 0.01 mg./liter and av. of 0.02. Reaction takes place in 2-3 hrs. at room temp., considerably reducing the 18 hrs. required with de Boer's indicator. With 100-mg. sample, max. sensitivity is obtained by addition of 5 ml. of indicator and 10 ml. 1/12 *N* hydrochloric acid. When concentration of fluorine is increased, color changes from blue to violet region of spectrum. Method can be used for determination of fluorine in water. Potassium, sodium, calcium, magnesium, chloride, bromide and nitrate have very slight influence, while aluminum, sulfate, phosphate,  $\text{AsO}_3$  and  $\text{AsO}_4$  influence color considerably. Ten references.—C. A.

**Picrate Method for the Determination of Phenol in Water.** M. P. BABKIN. *Lab. Praktika* 10: 22 ('37); *Chem. Zbl.* 2: 740 ('38). By means of the picrate method (reaction between phenol, nitric acid and sulfuric acid) as little as 1 mg. of phenol per liter of water can be determined colorimetrically. The accuracy of the determination is  $\pm 20\%$ .—W. P. R.

**Titration Theory of Dilute Solutions.** I. W. ARBATSKY. *Z. anal. Chem.* (Berlin) 115: 117 ('38). Theory of titrating dil. solns. is discussed with respect to magnitude of error and detn. of proper correction values. Titration of carbonates with phenolphthalein and  $\alpha$ -naphtholphthalein as indicators is discussed in detail and formulas for detg. correction factor are worked out. Advantages of latter indicator are pointed out with respect to this titration.—C. A.

**Determination of Traces.** *Ind. Eng. Chem.-Anal. Ed.* 11: 59 (Feb. '39). *Spectrographic Methods of Trace Analysis.* J. S. OWENS. Spectrographic methods are rapid, sensitive, accurate, free from judgment errors, and adaptable to analysis of variety of substances. Photographic photometry has increased usefulness of method. Amt. of material required is extremely small and preparation is simple. Precision and accuracy are great for traces of elements. *Fluorescent Analysis of Inorganic Materials.* C. E. WHITE. p. 63. Fluorescence is usually excited by rays in near ultraviolet range and used on solid, liquid or gaseous sample. May be used in titrations, e.g., iodine with sodium thiosulfate, using fluorescent indicators; or detn. of 0.2  $\gamma$  of aluminum in dilution of 0.1 p.p.m., using Pontachrome Blue Black R. *Isolation and Determination of Traces of Metals.* H. J. WICHMAN. p. 66. Using dithizone in chloroform soln., various metals, including copper, zinc and lead can be extracted from water soln., separated from each other and determined by appropriate methods. *Spectrophotometric Methods in Modern Analytical Chemistry.* S. E. Q. ASHLEY. p. 72. Spectrophotometric colorimetry, though neglected by most analytical chemists, has many advantages. Brief resumé is given of types of instruments available, errors and limitations of the method, and analytical results obtainable. *Present Status of Colorimetry.* M. G. MELLON. p. 80. Treatises of Yoe and of Snell and Snell give over 900

colorimetric methods using 700 different reagents for nearly 400 elements, radicals and compds. Methods of measurement are classified and suggestions are made for a consistent nomenclature. *The Use of Calomel for Separation, Detection and Estimation of Easily Reduced Elements.* GORDON G. PIERSON. p. 86. Using selective pptn. conditions, calomel may be used for detection and estimation of several elements, including iodine. In 5 ml. of cold, very slightly acid test soln., 0.01 mg. of iodine gives yellow color on 0.1 gm. of calomel.—*Selma Gottlieb.*

**Photoelectric Methods in Analytical Chemistry.** RALPH H. MÜLLER. Ind. Eng. Chem.-Anal. Ed. **11**: 1 (Jan. '39). A comprehensive review of photoelectric photometry with bibliography of 239 titles, including 30 dealing with applications to water analysis and control.—*Selma Gottlieb.*

**Spectrographic Analysis of La Plata River Water.** ROBERTO F. RECORDER. Bol. Obras Sanitarias Nacion (Buenos Aires) **3**: 16 (Jan. '39). Semi-quantitative spectrographic examinations of raw and treated water samples taken from the Buenos Aires supply, were made by the Physics Institute of La Plata University to supplement the information given by the usual analytical procedures for the determination of mineral substances in water. Arc and spark spectrograms were obtained for each sample of water. The quartz spectrograph used allowed a total spectrum length of 90 cm. covering the region from 2300 to 8000 Angstrom units. Platinum and iridium lines appeared in the spectrum of samples dried in platinum dishes. No such trouble was experienced when residues obtained in quartz dishes were examined. Certified carbon electrodes, containing slight traces of impurities with identified lines, were used in all the examinations. An electrolytic iron electrode of known spectrum was used as comparison electrode. The wave length of each spectrum line given by the water samples was determined by lineal interpolation between the wave lengths of two known lines in the reference iron spectrum located one on either side of the unknown line. The spectrograms showed the presence in the raw water of calcium, magnesium, iron, silicon, aluminum, sodium, manganese, potassium, strontium, zinc, lithium, titanium, cesium and fluorine. (Calcium fluoride band.) The same elements, with the exception of cesium, and in addition copper and lead were found in the treated water. The difference in the mineral constituents of these waters is believed to be the result of treatment and circulation through the distribution system.—*J. M. Sanchis.*